SEWAGE AND INDUSTRIAL WASTES

(Formerly Sewage Works Journal)

JOURNAL of the FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS

Special Features

Two-Stage Chlorination—Pierce

Vacuum Filter Theory—Halff

Cotton Finishing Wastes Treatment—Bogren

Salmonella in Irrigation Water—Dunlop, et al.

AUGUST, 1952—VOL. 24, No. 8

Make Plans to Attend

SILVER ANNIVERSARY MEETING

OF THE

FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS

IN CONJUNCTION WITH

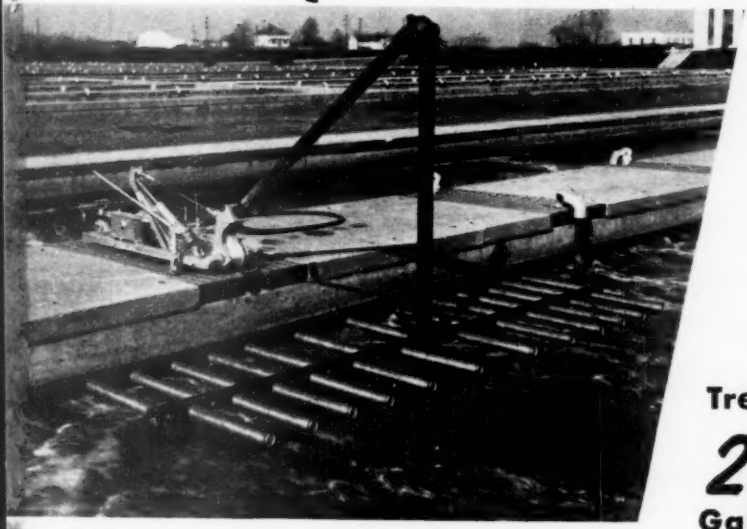
NEW YORK SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

HOTEL STATLER
NEW YORK CITY

October 6-9, 1952

FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS
325 ILLINOIS BUILDING CHAMPAIGN, ILLINOIS

Chicago SWING DIFFUSERS Proven IN 261 PLANTS



Treating More Than

2½ Billion

Gallons Per Day...

COLUMBUS, OHIO

Replacement of plates with Chicago Swing Diffusers made possible the efficient aeration of 17,000,000 more gallons per day (48,000,000 to 65,000,000). This was accomplished without adding to existing tanks or blowers.

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Installation of Chicago wide-band aeration in place of existing aeration system practically doubled aeration capacity without additional tanks. (10,000,000 to 19,210,000).

Over the past six years the engineers for these major plants have specified Chicago Swing Diffusers . . .

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Hunts Point Plant
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Complete list of installations and consulting engineers available upon request.

Leading consulting and design engineers and plant operators agree that Chicago Swing Diffuser Systems offer important exclusive advantages for new plants. And, in addition, they provide a logical solution where existing aeration systems are inadequate. Here are a few reasons why . . .

ACCESSIBILITY . . . Individual swing units are easily raised for inspection and diffuser cleaning.

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ECONOMICAL OPERATION . . . Lowest cost simplified cleaning restores new tube performance, permitting maintenance of lower blower pressures.

EFFICIENT AERATION . . . Wide-band air diffusion provides greater contact volume of diffused air with tank liquor and the longest contact period. No unaerated center core.

Complete operating histories and test data on Chicago Diffuser Systems are available for your review.

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622 DIVERSEY PARKWAY — **CHICAGO 14, ILLINOIS**

Flush Kleen, Scru-Peller, Plunger, Horizontal and Vertical Non-Clogs Water Seal Pumping Units, Samplers.

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SEWAGE AND INDUSTRIAL WASTES

REG. U. S. PAT. OFF.

A monthly Journal devoted to the advancement of fundamental and practical knowledge concerning the nature, collection, treatment and disposal of sewage and industrial wastes, and the design, construction, operation and management of such works.

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There are three ways to handle open flow:

Kennison Nozzle

1. Leave it unmetered.
2. Build a comparatively expensive concrete flume with accuracy dependent upon precise formwork.
3. **Install a Builders Kennison Nozzle . . . the *sure* way to get accurate open flow metering under the widest range of flow conditions.**

Here are a few of the reasons why so many sewage, waste treatment, and industrial plants are finding Builders Kennison Nozzle the ideal way to meter open flow! This scientifically designed nozzle comes ready for installation. Accuracy, built-in at the factory, is not dependent upon precise forms, structures, spillways, channels, weirs, or flumes. The special shape of this nozzle results in a very efficient rating curve, insuring accurate metering at both high and low flow rates. In handling solids-bearing liquids, the Kennison Nozzle has a self-scouring action which helps keep maintenance to a minimum.

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This type of meter is designed to operate under two maximum differential pressure heads. The first, 114.4" of water, permits measurement over a 13 to 1 range while the second, 64.5" of water, provides for a 10 to 1 measuring range. Standard meter design includes indicating, recording and totalizing mechanisms.

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SIMPLEX

VALVE AND METER COMPANY

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Association	*Place*	*Time*
Pennsylvania Sewage and Ind. Wastes Association	Pennsylvania State College, State College, Pa.	Aug. 27–29, 1952
Kansas Sewage Works Assn.	University of Kansas, Lawrence, Kans.	Sept. 3, 1952
Iowa Sewage Works Association	Grinnell College, Grinnell, Iowa	Sept. 9–11, 1952
Rocky Mt. Sewage Works Assn.	Frontier Hotel, Cheyenne, Wyo.	Sept. 15, 1952
Kentucky-Tennessee Industrial Wastes and Sewage Works Assn.	Andrew Johnson Hotel, Knoxville, Tenn.	Sept. 15–17, 1952
South Dakota Water and Sewage Works Conference	St. Charles Hotel, Pierre, So. Dakota	Sept. 17–19, 1952
German Sewage Technologists Assn.	Weisbaden, Germany	Sept. 17–20, 1952
Mo. Water and Sewerage Conference	Governor Hotel, Jefferson City, Mo.	Sept. 21–23, 1952
Georgia Water and Sewage Association	Georgia Inst. of Technology, Atlanta, Ga.	Sept. 24–26, 1952
North Dakota Water and Sewage Works Conference	Gladstone Hotel, Jamestown, N. D.	Oct. 2–3, 1952
West Virginia Sewage and Industrial Wastes Assn.	Chancellor Hotel, Parkersburg, W. Va.	Oct. 3–4, 1952

SILVER ANNIVERSARY MEETING

Federation of Sewage and Industrial Wastes Associations
in conjunction with
New York Sewage and Industrial Wastes Association
Hotel Statler, New York City
October 6–9, 1952

Pacific Northwest Sewage and Industrial Wastes Assn.	Multnomah Hotel, Portland, Ore.	Oct. 23–24, 1952
The Canadian Institute on Sewage and Sanitation	Seigniory Club, Montebello, Que.	Nov. 3–5, 1952
New England Sewage and Industrial Wastes Assn.	Taunton, Inn Taunton, Mass.	Nov. 5, 1952
North Carolina Sewage and Industrial Wastes Assn.	Skyland Hotel, Hendersonville, N. C.	Nov. 10–12, 1952
Florida Sewage and Industrial Wastes Association	San Carlos Hotel, Pensacola, Fla.	Nov. 16–19, 1952

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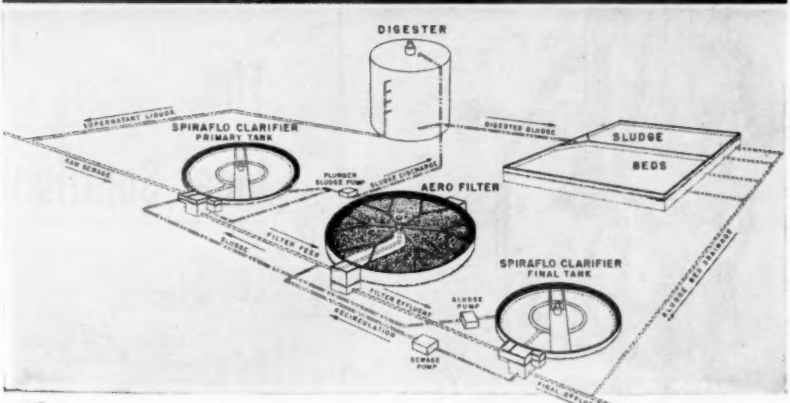

THIS LOW RATE, HIGH CAPACITY TRICKLING FILTER PLANT IS ESTABLISHING AMAZING RECORDS FOR HIGH PURIFICATION AT EXCEPTIONALLY LOW COST

These figures show a typical result produced by a Yeomans installation of the "Aero-Filter" and "Spiraflo" Clarifier in combination.

Overall plant efficiency is always higher when these units are used together. The method is especially effective for treating strong industrial wastes.

OPERATIONAL DATA . . . SPIRAFLO—AERO-FILTER PLANT *

1949 Date	Sewage Flow M.G.D.	Raw Sewage		Primary Effluent				Final Effluent				Overall Plant Reduction	
		Sus. Solids PPM	B.O.D. PPM	Sus. Solids PPM	Per Cent Reduction	B.O.D. PPM	Per Cent Reduction	Sus. Solids PPM	Per Cent Reduction	B.O.D. PPM	Per Cent Reduction	Sus. Solids Per Cent	B.O.D. Per Cent
6-15	0.2996	210	255	79	62.4	136	46.6	28	64.6	18	86.8	86.6	93.0
6-16	0.2467	425	260	85	80.0	127	51.2	23	73.0	19	85.1	94.6	92.6
6-17	0.2554	410	235	124	69.7	141	40.0	28	77.4	18	87.3	93.3	92.4
6-18	0.2487	308	222	76	75.4	108	51.4	34	55.3	16	85.2	89.0	92.8
6-19	0.1753	222	218	64	71.2	132	39.5	17	73.5	—	—	92.4	—
6-20	0.2666	262	320	88	66.4	147	54.1	20	77.3	30	79.6	92.4	90.6
6-21	0.2279	240	215	78	67.5	138	35.8	19	75.7	20	85.7	92.0	90.7
Average	0.2456	297	247	85	70.5	132	45.5	24	71.0	20	85.0	91.5	92.0

* Name of plant furnished on request, as well as operating data from other installations.

"Spiraflo" Advantages

- Increased B.O.D. and settleable solids removal —influent is introduced tangentially into annular race around the periphery. Sewage spirals downward slowly and enters main clarification compartment at the bottom of the tank. Uniform, slow diffusion of waste under the skirt results in greatly reduced velocities and provides maximum removal of settleable solids.
- Greater removal of finely divided particles—due to upward flow through sludge blanket. This increases flocculation and results in more effective removal of finely divided particles before waste enters the effluent weirs.
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"Aero-Filter" Advantages

- Thorough uniform distribution—therefore costly recirculation for dilution is unnecessary
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FOR INDUSTRIAL WASTES, TOO . . . Many types of industrial wastes can be successfully treated by direct application to the Aero-Filter without preliminary sedimentation.

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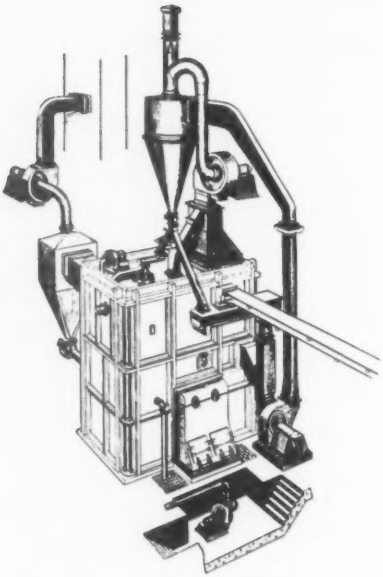

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C-E systems now in service are meeting the requirements of both large and small communities – serving equivalent populations from 6,000 to 3,600,000. The list below shows the most recent installations.

C-E Raymond Systems are flexible in layout, highly efficient and fully reliable; they provide for maximum utilization of waste heat. They mean for any community the end of stream or atmospheric pollution resulting from the disposal of sewage wastes.

The services of C-E specialists are available to help you plan for the most practical and economical answer to your sludge disposal problems. For prompt attention, get in touch with one of the Combustion offices listed below.

C-E Raymond Systems installed, under construction or on order since 1945

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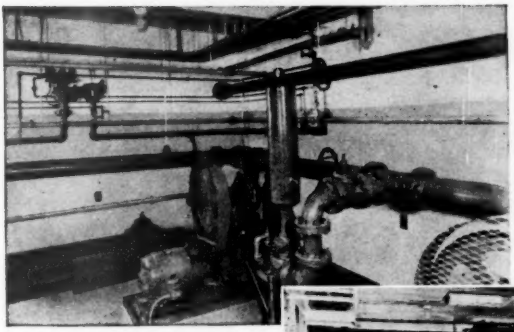

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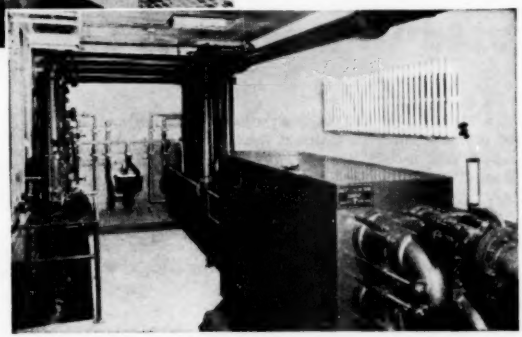

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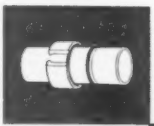

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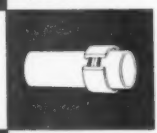

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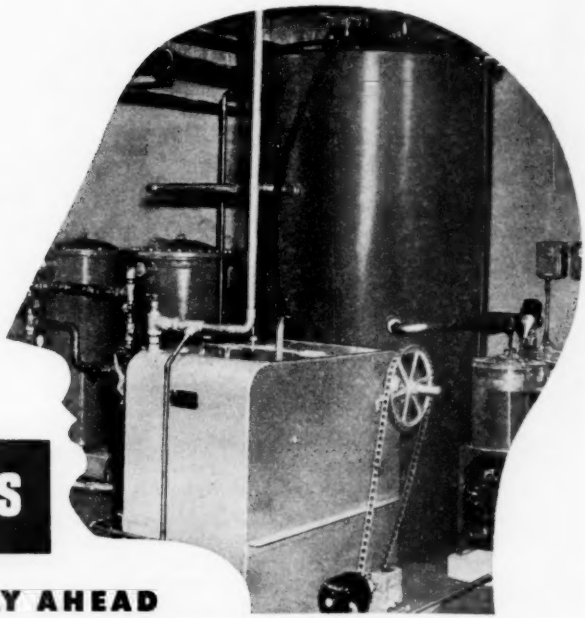

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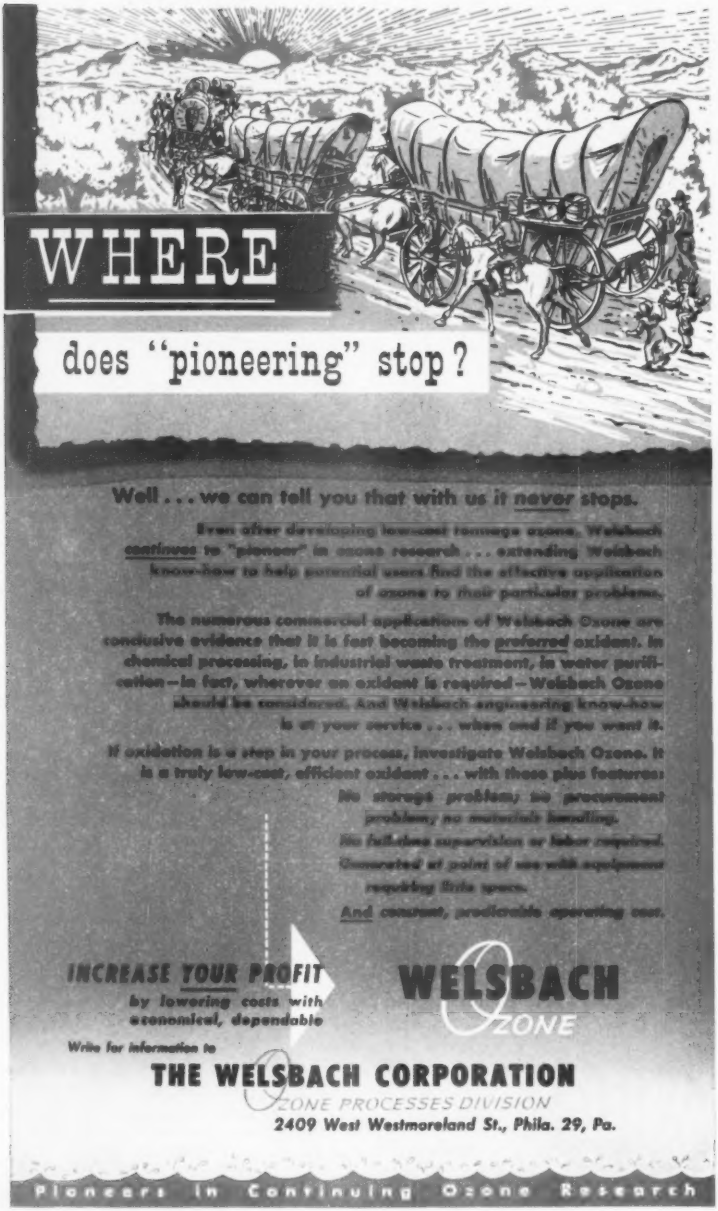
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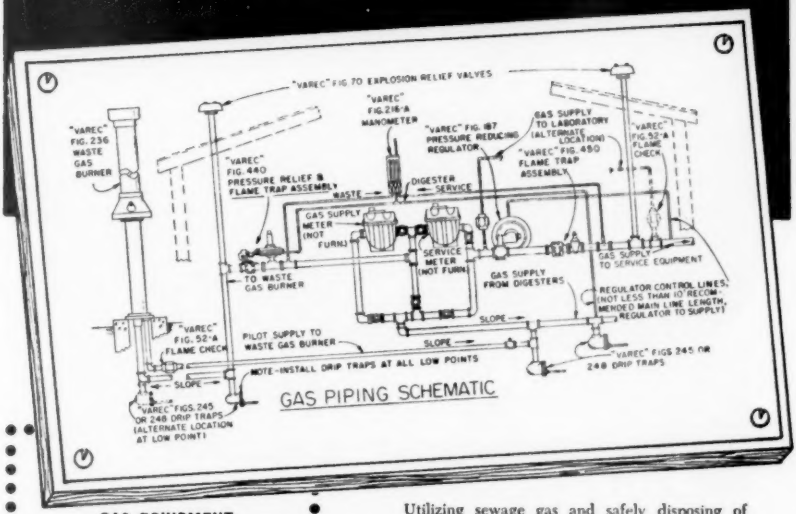
"VAREC" FIG. 70 EXPLOSION RELIEF VALVES
"VAREC" FIG. 236 WASTE GAS BURNER
"VAREC" FIG. 216-A MANOMETER
"VAREC" FIG. 187 PRESSURE REDUCING REGULATOR
GAS SUPPLY TO LABORATORY (ALTERNATE LOCATION)
"VAREC" FIG. 450 FLAME TRAP ASSEMBLY
"VAREC" FIG. 52-A FLAME CHECK
"VAREC" FIG. 440 PRESSURE RELIEF & FLAME TRAP ASSEMBLY
WASTE
DIGESTER SERVICE
GAS SUPPLY METER (NOT FURN.)
SERVICE METER (NOT FURN.)
GAS SUPPLY TO SERVICE EQUIPMENT
TO WASTE GAS BURNER
GAS SUPPLY FROM DIGESTERS
REGULATOR CONTROL LINES: (NOT LESS THAN 10 RECOM. MENDED MAIN LINE LENGTH, REGULATOR TO SUPPLY)
"VAREC" FIG. 52-A FLAME CHECK
PILOT SUPPLY TO WASTE GAS BURNER
SLOPE
SLOPE
"VAREC" FIGS. 245 OR 248 DRIP TRAPS
NOTE-INSTALL DRIP TRAPS AT ALL LOW POINTS
SLOPE
"VAREC" FIGS. 245 OR 248 DRIP TRAPS (ALTERNATE LOCATION AT LOW POINT)
GAS PIPING SCHEMATIC

GAS EQUIPMENT SPECIFICATIONS

a. **PRESSURE RELIEF AND FLAME TRAP ASSEMBLY**
"VAREC" Fig. No. 440

b. **FLAME TRAP ASSEMBLY**
"VAREC" Fig. No. 450

c. **FLAME CHECK**
"VAREC" Fig. No. 52A

d. **EXPLOSION RELIEF VALVES**
"VAREC" Fig. No. 70-1

e. **WASTE GAS BURNER**
"VAREC" Fig. No. 236

f. **DRIP TRAPS**
"VAREC" Fig. No. 245 (automatic)
"VAREC" Fig. No. 248 (hand operated)

g. **MANOMETERS**
"VAREC" Fig. No. 216A

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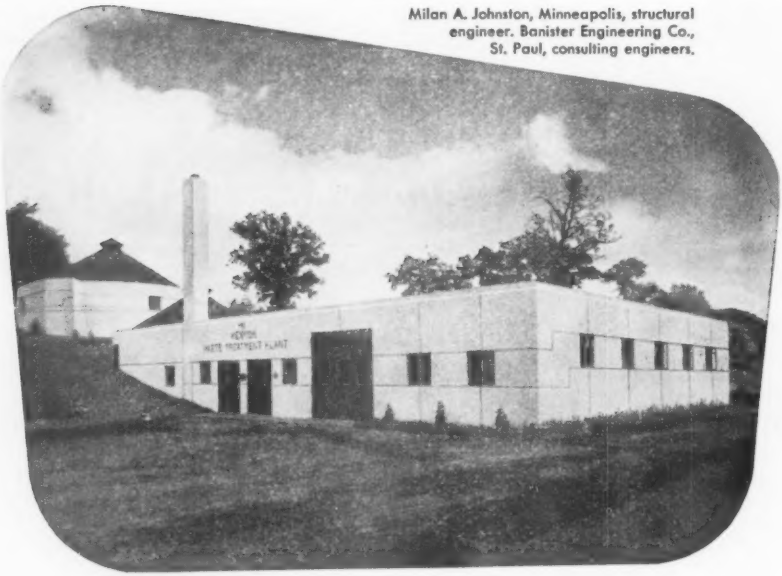

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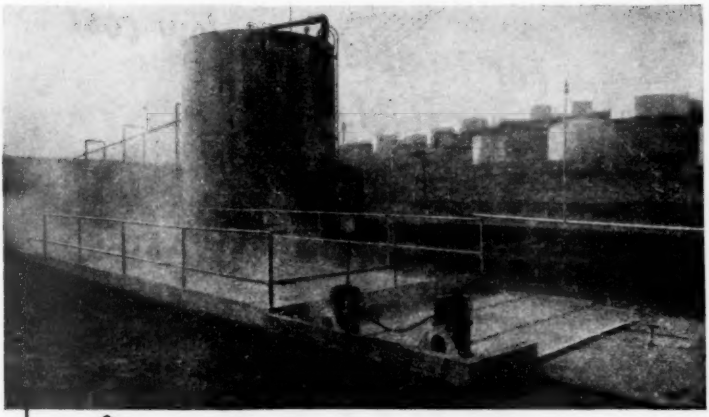

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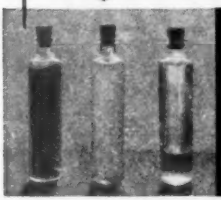

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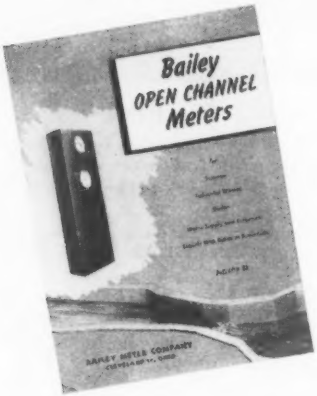

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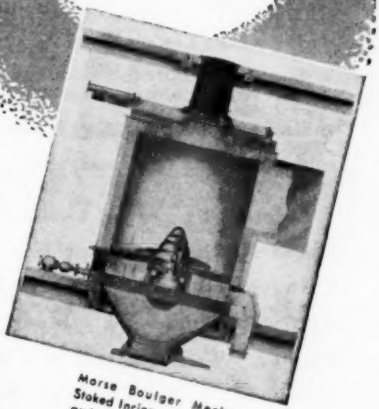

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SEWAGE AND INDUSTRIAL WASTES

Journal of the Federation of Sewage and Industrial Wastes Associations

Volume 24 **August, 1952** **Number 8**

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Proceedings of Member Associations

Montana Sewage and Industrial Wastes Association 326a

New York Sewage and Industrial Wastes Association 326a

W. H. Wisely, *Editor*, 325 Illinois Building, Champaign, Illinois

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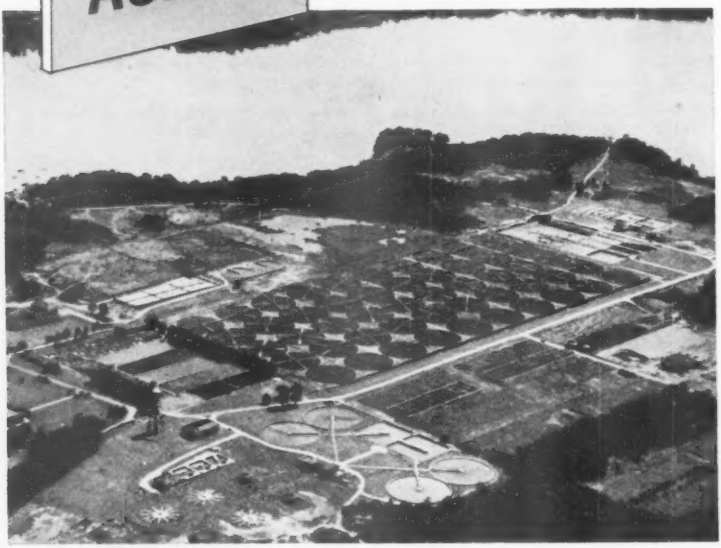
30 ACRES of trickling filters at Baltimore's Back River Plant...



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Sewage Works

TWO-STAGE CHLORINATION—AN EFFECTIVE AND PRACTICAL METHOD OF SEWAGE DISINFECTION

By Donald M. Pierce

Sanitary Engineer, Michigan Dept. of Health, Lansing, Mich.

I. Preliminary Investigation

Two-stage chlorination has been demonstrated to be a practical, dependable method of disinfection to consistently produce an effluent of low coliform density at two primary treatment plants in Michigan. Sufficiently extensive observations were made at the Traverse City plant during two summer seasons to indicate its marked superiority over single-stage chlorination, either preceding or following sedimentation. By simultaneous dosage of chlorine at these two points, it was demonstrated that a coliform density of not exceeding 500 M.P.N. per 100 ml. could be obtained about 98 per cent of the time with ordinary effort. Similarly, it was rather well-established that the effluent would meet a standard of 100 more than 90 per cent of the time and of 36 more than 80 per cent of the time with a contact period of about 15 min. for the post-chlorination phase. The ease of consistent maintenance of a chlorine residual within reasonable limits at all times with no special or unusual equipment was evidenced by the finding that a residual of some magnitude, but less than 2.0 p.p.m., was maintained 100 per cent of the time at this plant while two-stage chlorination was employed. The results obtained at the Imhoff tank plant at Holland, Mich., generally confirmed the Traverse City observations.

Because the plant facilities and processes, both as to type and size of installation and the sewage treated, are rather typical of a great many others throughout the country, it is believed that the methods employed at these plants may be equally adaptable at many plants and may produce equally favorable results.

The studies discussed in this report include a brief exploratory survey conducted by the technical staffs of the Michigan State Department of Health and Stream Control Commission at primary treatment plants at Traverse City, Charlevoix, and Petoskey, Mich., in the summer of 1948 to find out how effective were current methods of disinfection customarily employed at these plants.

In the following summer the studies were continued by the two staffs at Traverse City to determine the practicability of two-stage disinfection for dependable, consistent production of effluents of very low coliform density. The method consisted of adding chlorine to the raw sewage ahead of a grit chamber at a rate sufficient to produce a residual of some magnitude, if possible, but less than 0.5 p.p.m., and the simultaneous addition of chlorine to the settling tank effluent so as to produce a residual of about 0.5 p.p.m. after a minimum contact period of 5 min. for every particle. The results of both chlorine dosage control and level of coliform density were so gratifying that these methods, including bacteriological testing, were continued during the summer of 1950 by the regular

plant operating staff. At Holland and Muskegon a similar program was adopted, along with studies on single-stage post-chlorination for comparison.

The process of two-stage, or split chlorination as it is usually termed, is not a new concept of disinfection. It has been employed occasionally at many plants throughout the world, but has not generally been regarded with as much favor as single-stage post-chlorination, or even pre-chlorination. The reasons for this are rather obscure, for the method is seldom mentioned in the technical literature except in occasional short references to its use or the generalized statement that it would produce lower coliform densities than chlorination at a single point or that it is somewhat more expensive than the single-stage method used. It may be deduced from a study of the regulations of many of the state regulatory agencies and from a review of the technical literature that chlorination of plant effluents ahead of a contact tank or outfall sewer is the most popular and generally accepted method of disinfection, regardless of the coliform density which local conditions demand. This process, in spite of its recognized limitations, is known to reduce coliform densities sufficiently to meet certain conditions where rigid requirements are not warranted. Whether or not effluent requirements such as 500 M.P.N. per 100 ml. can be consistently met by single-stage disinfection is not found by the author either by field observation or review of the technical literature.

Chlorine Demand Variations

It is well-established that dependable consistent disinfection of sewage to meet any standard of low coliform density is difficult of attainment by generally accepted methods. This is not at all surprising when one considers the inconsistent and unpredictable nature of sewage. Frequent discussions are found in the literature on the variation in the chlorine demand of sewage, the term generally connoting the rate of chlorine dosage, expressed in p.p.m., required to produce a chlorine residual of 0.1 p.p.m. after a contact period of 15 min. In 1927 Tiedeman (1) reported that the chlorine demand of settled sewage of essentially domestic origin at the Huntington, N. Y., plant varied from 2 to 10.6 p.p.m. in a single day, with daily averages of about 6.8 p.p.m., and seasonal demands ranged from daily avrages of 6.5 to 13 p.p.m. Symons (2) in his studies of the Buffalo, N. Y., sewage found that, on the average, the normal variation in chlorine demand was from 70 to 130 per cent of the daily average during an average day, with single peak variations of 800 to 1,000 per cent of the average for a single day occurring infrequently. Other investigations have reported wide ranges in chlorine demand, sometimes greater and sometimes less than these, but always different for each sewage and period of time.

Many of the factors influencing the required chlorine dosage are well-known, but their absolute or interrelated influence on the demand is more difficult to determine. Scott and Van Kleeck (3) observed that significant factors influencing the rate of chlorine dosage to satisfy the demand include rate of sewage flow; the strength, freshness, and temperature of the sewage; the range in variations in concentration; content of industrial wastes; effects of intermittent discharge of grit chamber washings or digestion tank overflows or sludge bed underdrainages; chlorine contact period; method of application of chlorine; and the contact of chlorine and sewage with decomposing sludge and scum. Extensive studies of Rudolfs and Gehm (4) indicate many of the complex factors which influence the frequent and often unpredictable variation of the dosage required.

To some extent elimination of the factor of variation in sewage flow is helpful in providing the required dosage. This is sometimes accomplished by chlorine feeding equipment, which automatically proportions the dosage rate to the rate of sewage flow. It is well-established, however, that this control of itself is inadequate. It is usually still necessary to make manual adjustments, sometimes very frequently and often too late, when chlorine is applied at a single point. If, however, it is applied at two points, much greater opportunity is afforded for correction.

An interesting and apparently successful method of dosage control employed at the Buffalo, N. Y., sewage treatment plant is reported by Symons and Simpson (5). The procedure consisted of frequent manual adjustments of the chlorinators based upon predicted patterns of demand, with further adjustments on the basis of observed chlorine residuals. Chlorine was added to the raw sewage ahead of sedimentation tanks having detention periods of 45 min. or more. A sample of the well-mixed chlorinated sewage was collected within 2 min. after initial mixing and the sample held for 15 min., after which the chlorine residual was determined. Adjustment in the dosage rate was made accordingly. The basic hour-to-hour dosage rate pattern was determined by consideration of similar previous conditions. This method is perhaps more effective than any in current general use for single-stage disinfection.

Other Previous Data

Although frequent mention of the difficulty of controlling dosage rates to produce a residual after stipulated periods of contact is found in the literature, very little information is found on the hour-to-hour chlorine residual records of actual plant operation. Records of operation at several plants examined by the author reveal that chlorine residuals are frequently lacking with single-stage disinfection, whether for pre-chlorination or post-chlorination. These observations have been confirmed by several other observers.

A search of the technical literature also revealed very little information on the hour-to-hour, day-to-day plant records of coliform density for any method of chlorination. The most obvious reason for this lack of information is that relatively few operators conduct routine analyses for the determination of coliform density. The majority of the references to work of this kind are based upon special investigative studies and are often presented as statistical evaluations rather than a series of actual values for each sample. There are, however, sufficient references to provide some general concept of the extent of disinfection accomplished by various methods under a few conditions. Some basic investigations on the relationship of chlorine residuals, contact periods, and coliform densities were reported by Tiedeman (1) on his studies at the Huntington, N. Y., plant. On the basis of one sample per month of settling tank effluent collected over a period of about a year, where each sample was subjected to varying periods of contact, he concluded that a 5-min. reaction period was sufficient when a chlorine residual of 0.2 to 0.6 p.p.m. by orthotolodine was maintained. He recommended, however, that a 15-min. period of contact be provided with chlorine residuals of this magnitude, but did not state what level of coliform density may be consistently assured thereby. These observations were in part confirmed by Symons and associates (2) (5) in their exhaustive studies of plant performance at Buffalo, N. Y., during which they conducted 11,000 sets of bacteriological analyses. Chlorination of the raw sewage was closely controlled so as to produce a residual of 0.1 p.p.m. after 15

min. of the total of 45 or more minutes of contact in the settling tanks. During the 31 months of this study, the average coliform density of the raw sewage and settling tank effluent were 7,150,000 and 104,000 M.P.N. per 100 ml., respectively. This represented an over-all reduction of about 98.6 per cent. These values represent the 31-month average of the monthly averages of samples collected daily at 5 periods per day. Daily sampling periods were carefully chosen to represent the 24-hr. average. Monthly averages of coliform densities of the effluent for dry-weather periods ranged from a low of 1,000 to a high of 359,000 M.P.N. per 100 ml. No data are found on coliform densities of individual samples.

Other significant and valuable information on chlorination procedures includes the reported studies of Eliassen, Heller, and Krieger (7). Statistical procedures are formulated for the development of a rational basis for control of chlorination by residual chlorine to produce desired coliform densities in the plant effluent. The basic experimental work was conducted on a weak domestic sewage from one of the sewers of the Borough of the Bronx, New York City. The sewage was filtered through nonabsorbent cotton to simulate settled sewage, after which chlorine was added in various percentages of the demand and subjected to slow or rapid mixing or to no mixing at all, followed by a contact period of 10 min. The importance of thorough mixing is demonstrated. It is interesting to note the finding that chlorine dosage sufficient to maintain a residual of 0.55 p.p.m., after thorough mixing and 10-min. contact, was required in order to produce coliform densities of 300 M.P.N. per 100 ml. 50 per cent of the time. It is perhaps of equal interest to note that coliform densities, where adequate mixing was employed and where chlorine residuals varied from 0.35 to 0.6 p.p.m., ranged from less than 2.2 to 25,000 per 100 ml. with 20 per cent at 2,500 or above. Much lower efficiencies were reported where no mixing was provided prior to the 10-min. contact period.

The magnitude of chlorine residual required to consistently maintain a stipulated coliform density of extremely low order in single-stage post-chlorination is discussed by Heukelekian and Day (8). In their studies at four plants they found that an ortho-tolodine residual of 0.2 to 0.5 p.p.m. after 15-min. contact time will meet an effluent standard of 250 per 100 ml. 50 per cent of the time, and the same residual with 30-min. contact time will produce this coliform density 80 per cent of the time. They further report that compliance with this effluent standard increases with higher residuals until, with 2.1 to 4.0 p.p.m. ortho-tolodine residual, 100 per cent compliance is obtained with a 10-min. contact time. It is evident that in routine plant operation much higher chlorine residuals than these would have to be maintained much of the time in order to assure that the residual would not drop below the required level, unless chlorine feed were regulated automatically with relation to residuals.

The accumulated experience and observations as reported by the investigators mentioned and by many others concerned with chlorination problems and practices seem to confirm the observations of the author and others in the state regulatory agencies in Michigan that consistent production of very low coliform densities in the effluent of primary treatment plants is most difficult, if possible, to achieve with usual single-stage chlorination methods. It is equally evident from the studies discussed in this paper that close controls over chlorine residuals are necessary in two-stage disinfection to attain this objective.

It is realized, of course, that the results obtained in these studies may

TABLE I.—Summary of Plant Conditions During Single-Stage Chlorination Studies

Item	Charlevoix	Petoskey	Traverse City
Est. population served	5,000	7,000	20,000
Survey period[1]	10 AM–4:30 PM	10 AM–5 PM	9 AM–4 PM
Range of flow (m.g.d.)	0.4–0.55	0.6–0.9	2.7–4.0
Sewer system:			
Sep. or combined	Separate	Separate	Partly each
Gravity or pumped	Gravity	Gravity	Pumped
Grit removal	Manual	None	Mechanical
Screening	Mech. shredded	Bar screen	Bar screen
Theor. displacement time:			
Settling tanks (min.)	190–245	130–185	95–135
Contact tank (min.)	190–245	28–40	11–16
Chlorinator capacity (lb./day)	100	80	560
Chlorine dosage range:			
Lb./24 hr.	62–80	75–80	125–350
P.p.m.	17.6–23.7	10.7–15.8	5.5–13.9

[1] Rates of sewage flow and chlorine dosage are for period of day indicated.

not be duplicated by precisely similar methods at other plants. The methods used, however, appear to have sufficient promise to warrant further studies at other installations. It is hoped that additional investigations may be undertaken to augment the work here described. Certainly there is a real and urgent need in many places for the maximum degree of disinfection within the practical limitations of attainment.

Single-Stage Chlorination Studies

Exploratory surveys of existing disinfection methods at three primary sewage treatment plants were conducted in August, 1948, by the Michigan Department of Health and the Michigan Stream Control Commission,* with the cooperation and assistance of the plant operating staffs. The plants are all municipally owned and operated, serving the cities of Traverse City, Charlevoix, and Petoskey, Mich. All are located along the shores of Lake Michigan or nearby tributary waters. In each case the plant effluent discharges into public waters used extensively for recreation, including swimming.

In several respects the three plants selected for study were known to be rather ideal for the proposed studies, because each was believed to be largely uninfluenced and unaffected by particularly unusual factors and circumstances which make disinfection control so difficult in many plants. The virtual absence of industrial wastes, the rather typical quality of the sewage and public water supply, and the rather typical kinds and capacities of plant treatment facilities and proc-

* Succeeded by the Michigan Water Resources Commission in 1949.

TABLE II.—Partial Chemical Analysis of Municipal Water Supplies in Survey Cities

City	Source	Total Solids (p.p.m.)	Fe (p.p.m.)	SO_4 (p.p.m.)	HCO_3 (p.p.m.)	Total Hardness, as $CaCO_3$ (p.p.m.)
Traverse City	Lake Michigan	152	Trace	14.5	148.8	132
Petoskey	Wells	255	Trace	29.6	217.0	220
Charlevoix	Wells	254	0.30	24.8	260.0	232

TABLE III.—Partial Chemical Analysis of Raw and Settled Sewage, 1948 Studies

Plant	Period	B.O.D. (p.p.m.)		Susp. Sol. (p.p.m.)	
		Raw	Settled	Raw	Settled
Traverse City	July Av.	275	210	131	80
	July Min.	260	180	92	48
	July Max.	290	240	158	100
Petoskey	Aug. Av.	272	213	273	113
	Aug. Min.	180	160	216	76
	Aug. Max.	360	280	360	140
Charlevoix	Aug. Av.	250	175	267	73
	Aug. Min.	180	140	200	64
	Aug. Max.	320	200	312	84

esses and the size of the community served seemed to provide a quite ideal combination for a basic study of disinfection methods.

Furthermore, the three plants were located sufficiently close together to permit the use of one laboratory for all studies, without much sacrifice of time.

Description of Plant Facilities

Facilities at each plant included plain sedimentation with mechanical sludge removal, separate sludge digestion, and chlorination. Supernatant liquor from the sludge digestion tanks was returned to the raw sewage. Table I gives a brief summary of pertinent conditions prevailing during the survey.

Quality of Sewage

Some indication of the quality of the raw sewage may be noted from Tables II and III. The monthly average, minimum, and maximum values of 5-day B.O.D. and suspended solids are believed to represent fairly the general range in quality of this sewage, although no tests were performed during the survey.

Organization of Survey

The survey party consisted of one sanitary chemist and one sanitary bacteriologist. Two sanitary engineers frequently contacted the party, made surveys on the public waters into which the plant effluents discharged, and made arrangements with municipal officials and plant personnel to assure uninterrupted conduct of the survey.

All chemicals, media, glassware, an incubator, and necessary supplies were transported from the laboratories of the Michigan Department of Health to the laboratory of the Traverse City sewage treatment plant, headquarters for the survey of the three plants. The sampling schedule was as shown in Table IV.

TABLE IV.—Sampling Schedule

	Charlevoix	Petoskey	Traverse City	
			Pre-Chlorination	Post-Chlorination
Day of week	Tues., Fri.	Wed., Thurs.	Fri., Sat.	Mon., Thurs.
Period of day	10 AM–4:30 PM	10 AM–5 PM	8 AM–5 PM	9 AM–4 PM
Total samples	26	22	24	20

Survey Methods

Insofar as practicable, the survey was conducted without disruption of customary operation practice. However, adjustments of the manually operated chlorinators were made more frequently than usual to control chlorine residuals as closely as practicable to the desired 0.5-p.p.m. level in the effluent of the tank in which chlorination was accomplished.

Chlorine was introduced with the raw sewage and well mixed therewith before entering the settling tanks at Charlevoix and for two of the survey days at Traverse City. During the other two days at Traverse City and both days at Petoskey the chlorine was applied to the effluent from the sedimentation tanks before entering a baffled chlorine contact tank.

Two samples of the chlorinated effluent were collected simultaneously in all instances, one for chlorine residual and the other for determination of coliform density. Sterile bottles with capacities of 100 to 125 ml. were used for each purpose.

The bottle for bacteriological examination contained 0.1 ml. of 0.012N thiosulfate oven-dried in the bottle following sterilization. This dechlorinated sample was transferred at once to the bacteriologist. Three portions with four decimal dilutions were planted in lactose broth within 10 min. following collection. All samples except those collected on the last two days at Traverse City were subjected

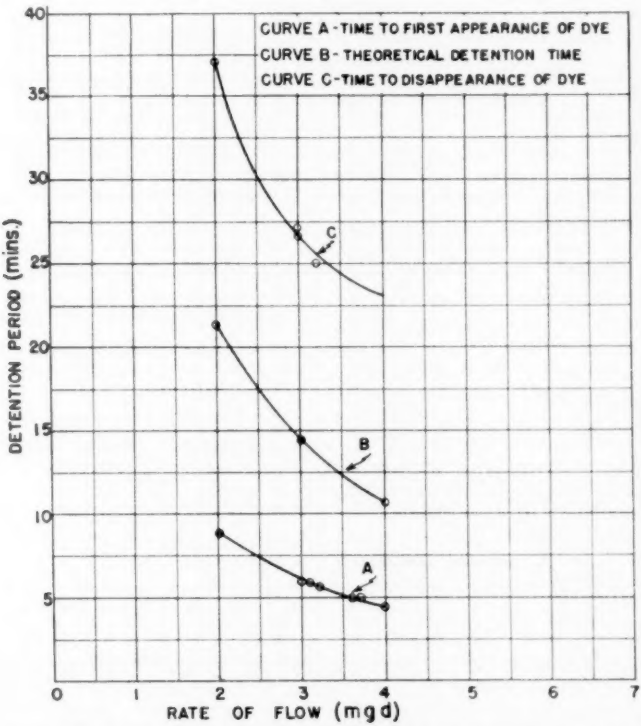

FIGURE 1.—Time of flow in chlorine contact tank, Traverse City, Mich.

to the confirmed test, using brilliant-green bile. All bacteriological work was performed in accordance with "Standard Methods" (6). On these last two days samples were carried through the presumptive test only, on the undoubtedly erroneous assumption that there would be no significant change in calculated coliform density by completion of the test. This assumption was not confirmed by data collected at the same plant during the more exhaustive survey in 1949 reported herein.

The sample for residual chlorine determination was collected in a plain sterile bottle. Residual chlorine was determined by the ortho-tolodine-arsenite method for water analysis prescribed by "Standard Methods" (6). Total, free, and combined chlorine were determined and recorded.

Fluctuations in Sewage Flow and Chlorine Requirements

To facilitate study of the variations in rates of sewage flow and chlorine requirements and the extent of effectiveness of chlorine dosage control to produce the desired level of chlorine

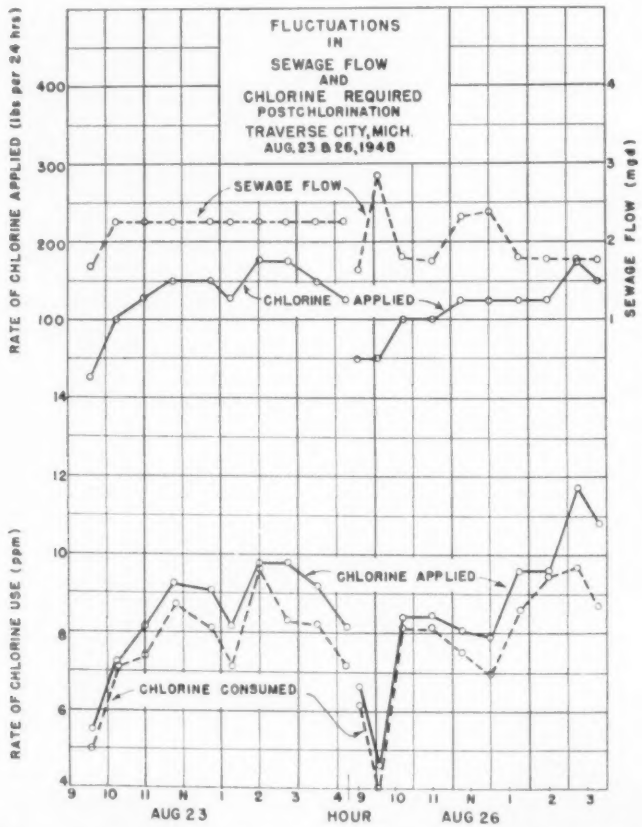

FIGURE 2.—Fluctuations in sewage flow and chlorine required for post-chlorination, Traverse City, Mich., August 23 and 26, 1948.

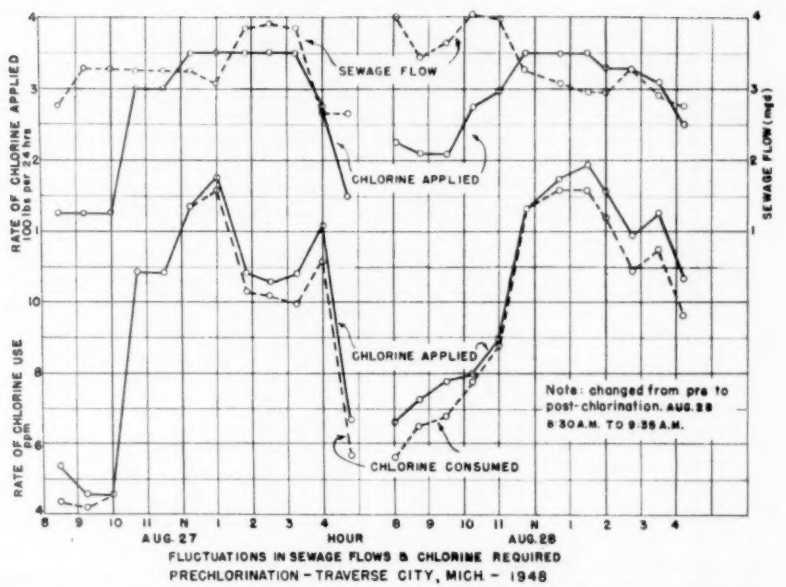
SEWAGE FLOW
CHLORINE APPLIED
CHLORINE APPLIED
CHLORINE CONSUMED
Note: changed from pre to post-chlorination. AUG. 28 8:30 A.M. TO 9:35 A.M.
RATE OF CHLORINE APPLIED 100 lbs per 24 hrs
RATE OF CHLORINE USE ppm
SEWAGE FLOW (mgd)
AUG. 27
HOUR
AUG. 28
FLUCTUATIONS IN SEWAGE FLOWS & CHLORINE REQUIRED
PRECHLORINATION - TRAVERSE CITY, MICH. - 1948

FIGURE 3.—Fluctuations in sewage flow and chlorine required for pre-chlorination, Traverse City, Mich., August 27 and 28, 1948.

residuals, such data for all samples collected at Traverse City and Charlevoix were plotted (Figures 1, 2, 3, and 4). The term "chlorine consumed" as here used represents the rate of chlorine applied (p.p.m.) minus the chlorine residual (p.p.m.). The magnitude of the chlorine residual then is the difference in these two ordinates.

The data for Petoskey are not shown because it was discovered shortly after the survey was begun that the maximum dosage rate of the chlorinator was insufficient to produce the desired 0.5-p.p.m. residual. The survey was continued in the hope that less chlorine might be required later on, but this did not happen, nor did the parts needed for expanding the chlorinator capacity arrive until after the survey was completed. This discovery was of immediate benefit to the operator of this plant both in demonstrating the need for more capacity, which had not been evident from the infrequent sampling customarily performed, and clearly showing the need for frequent adjustments in chlorine dosage required to compensate for changes in rate of sewage flow and in rate of chlorine demand per unit of sewage. Generally, the fluctuations in chlorine demand, as observed in the samples containing residuals, were of the same unpredictable nature both as to magnitude and time of occurrence as shown for the other two plants.

It may be noted from Figure 4 that a residual was observed in all samples on August 20 at Charlevoix, varying from 0.1 to 1.0 p.p.m. The highest rate of sewage flow for this day was 0.54 m.g.d., occurring at 10 AM. The corresponding chlorinator setting was 80 lb. per 24 hr. On the basis of these data and in the light of the experience of the operator it was assumed that the 80-lb. rate would be sufficient as long as sewage flows did not exceed this rate. That this assumption was erroneous

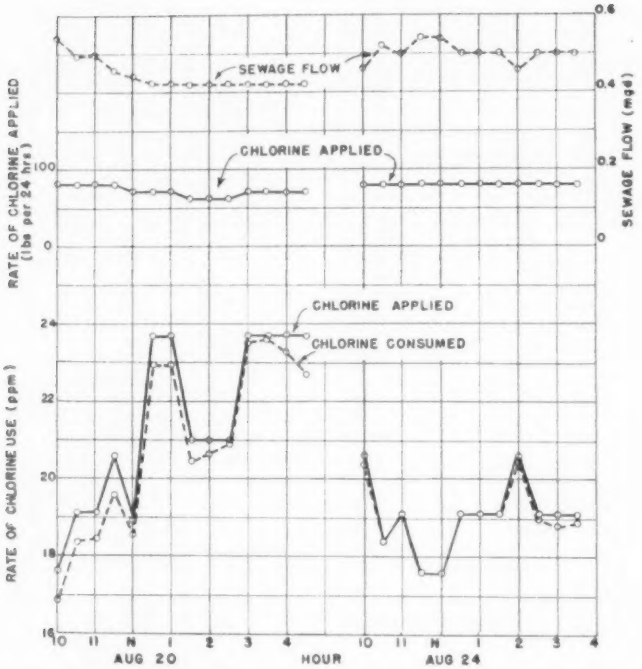

FIGURE 4.—Fluctuations in sewage flow and chlorine required for pre-chlorination, Charlevoix, Mich., August 20 and 24, 1948.

may be noted from the data for August 24. Of the 12 samples collected on that day, only 5 contained measurable residuals and all of these were 0.35 p.p.m. or less with continuous dosage rates of 80 lb. and flows never exceeding 0.54 m.g.d.

Similarily, a lack of relationship between magnitude of rate of sewage flow and the amount or rate of chlorine dosage to produce residuals may be noted in Figures 1, 2, and 3 for Traverse City, both for pre-chlorination and post-chlorination. Although rough trends from early morning lows to midday highs, followed by some falling off in demand in midafternoon, may be noted, estimates of the time of occurrence or magnitude of required dosage cannot be made, on the basis of experience, sufficiently close to permit control of chlorine residuals within practical limits. It is clear that a chlorinator with dosage proportioned automatically to sewage flow would not of itself afford good control. It would appear that effective dosage control can be best accomplished for pre-chlorination by imposing frequent adjustments upon a pre-estimated program of adjustments based on experience and observation, similar to that employed at Buffalo, N. Y., as described by Symons and Simpson (5). However, it would appear that such control requires a degree of precision and effort not practicable except in extensive installations. Ideally, perhaps, control could best be obtained by some automatic device translating instantaneous chlorine demand of the raw sewage into chlorine dosage requirements per unit of sewage flow

volume and with provision of automatic adjustment of chlorine dosage accordingly. Assuming that a residual of some magnitude is required for effective disinfection, and lacking this type of automatic control equipment, it may be reasoned that the objective can only be attained by frequent overdosage, often of substantial magnitude. When it is considered that such control might require overdosage of several parts per million, as indicated by these brief studies, it is evident that it would sometimes be difficult to follow because of the need for justification of what might appear to be a needless waste of chlorine.

Control of Coliform Density

The data secured on coliform densities were considerably weakened by frequent failure to maintain a chlorine residual in the effluent at the sampling point, both at Petoskey and Charlevoix. Because the samples for pre-chlorination at Traverse City were not subjected to the confirmed test, these data may not be directly compared with the pre-chlorination data for Traverse City or for Charlevoix, both of which are the results of the confirmed test. However, sufficient data were secured to warrant comparison from which some indication and inferences may be deduced.

Table V includes all samples at Traverse City and Charlevoix for which chlorine residuals of any magnitude were observed. Residuals varied from 0.05 to 2.0 p.p.m. for all three sets of data. No relationship between the magnitude of chlorine residuals and coliform densities was noted.

Because the coliform data for pre-chlorination at Traverse City are based on presumptive values it may be assumed that some further reduction in level would be produced by the confirmed test. Some further reduction in magnitude of coliform densities for Charlevoix would undoubtedly have been demonstrated by a better choice of dilutions, as 7 of the values were reported as less than 36,000 and 5 as less than 3,600.

TABLE V.—Comparison of Level of Coliform Density
Pre-chlorination *vs.* Post-chlorination

Coliform Density (M.P.N./100 ml.)	No. Samples Not Exceeding Coliform Density Specified		
	Traverse City		Charlevoix, Pre-Chlor.
	Post-Chlor.	Pre-Chlor.	
1,000	0	0	1
5,000	1	7	10
10,000	1	11	12
50,000	13	18	19
100,000	13	18	
240,000	19	19	
Total No. samples	20	19	19
Log. av. coliform density	45,000	4,600	4,300

These tabulated results indicate that much lower coliform densities were secured with long contact periods (2 to 4 hr.) following chlorination of raw screened sewage than with short contact periods of about 10 to 15 min. following plain sedimentation, when in all instances residual chlorine of the same general magnitude and generally less than 1.0 p.p.m. was present in the sample following these contact periods.

Summary of Observations

From these brief exploratory studies of chlorination at three rather typical small primary treatment plants largely uninfluenced by industrial wastes, the following indications were noted:

1. Fluctuations in chlorine requirements to produce a residual in the effluent, either with pre-chlorination or post-chlorination, are unpredictable both as to magnitude and time of occurrence, varying as much as 8 p.p.m. in a single day from 8:00 AM to 5:00 PM.

2. Chlorine residuals of any desired magnitude less than 1 p.p.m. are most difficult to control by ordinary and

customary methods in single-stage disinfection.

3. No definite trends or relationships were noted between the magnitude of coliform density and the amount of residual chlorine, when residuals varied between 0.5 and 2.0 p.p.m.

4. Substantially lower coliform densities were observed with chlorination of raw sewage followed by a 2- to 4-hr. theoretical contact period than with chlorination of settling tank effluent followed by theoretical contact periods of 10 to 15 min.

These indications of several deficiencies of both methods of single-stage disinfection for the consistent, continuous production of coliform densities of very low and predictable magnitude led to the decision to explore the merits of two-stage chlorination.

II. Plant-Scale Studies

Recognition of the limitations of single-stage chlorination of primary sewage plant effluents, as indicated by the 1948 studies both with respect to effluent quality and the impracticability of consistently maintaining any desired level of chlorine residual, led to the decision to explore the merits of two-stage chlorination as a means of disinfection control. The Traverse City plant was selected for these investigations, particularly because (a) the 1948 studies conducted there had established certain valuable basic information about the plant facilities and the quality of the sewage; (b) the facilities, both as to size and type, are typical of many plants in Michigan and elsewhere; and (c) some basis of comparison of single-stage and two-stage chlorination might be obtained.

During the summer of 1950, two-stage chlorination was continued at Traverse City as routine operation and the cities of Holland, Frankfort, and Muskegon, Mich., adopted the practices for the first time. Data from the Traverse City and Holland plants are included in the following discussion. At each plant chlorine was introduced with the raw sewage prior to sedimentation and again with the settling tank effluent ahead of a contact tank.

Traverse City Studies—1949

The 1949 investigations and studies were undertaken cooperatively by the Michigan Department of Health and the Michigan Water Resources Commission as in the previous summer. In planning the survey it was proposed that it be conducted over a period of sufficient duration, involving as many samples as practicable in order to reasonably establish, wherever possible, the merits and limitations of the process of two-stage chlorination.

The actual field investigations and studies were begun on June 22 and completed on July 18, 1949. The work was performed by two sanitary engineers, one from the Engineering Division of the Michigan Department of Health and the other from the technical staff of the Water Resources Commission. The operating staff of the plant materially assisted in the work by making adjustments in plant operation schedules and other cooperative efforts requested by the survey party.

Sewerage Facilities Involved

In most respects the investigations were conducted under conditions similar to those prevailing during the 1948 studies on single-stage chlorination. The major difference was the admission into the sewer system of wastes from cherry canning operations, during a portion of the survey period. Cannery operations began on about July 10 and considerably affected the quality of the sewage at the plant from July 12 to July 18, when the studies were terminated. These wastes origi-

nated largely from cherry washing and pitting operations. The sewage frequently contained whole cherries or parts thereof, slimes, and juices in solution. Aside from these wastes, no significant industrial wastes were known to be tributary to the sewer system connected to the plant.

Survey Methods

During the entire survey, except for a few hours when the chlorine supply was nearly depleted, chlorine was continuously introduced with the raw sewage in the grit chamber ahead of the four parallel plain sedimentation tanks. Chlorine was also added to the settling tank effluent and well mixed before entering the chlorine contact chamber. During several periods, usually beginning at about 8:00 AM and lasting for about 2 hr., chlorine was not applied at the second point when residuals of 1.0 p.p.m. or more were observed in the settling tank effluent. Control over the rate of chlorine feed was exercised with the purpose of maintaining some measurable residual at the points of discharge of the settling tanks and a 0.5-p.p.m. residual at the point of discharge of the contact chamber. Two 300-lb. manually operated solution feed chlorinators were used.

Samples were collected on 17 days, usually from about 8:00 AM to 3:30 PM at intervals of about 45 min. A total of 174 pairs of samples were collected from the settling tank effluent and an equal number from the effluent of the chlorine contact tank. One of each pair was tested for residual chlorine and the other for coliform density. With the exception of the 66 samples of contact tank effluent collected during the last 6 days of the survey, all samples for bacteriological analysis from both points were dechlorinated immediately after collection in 100- to 125-ml. bottles containing 0.1 ml. of sodium thiosulphate oven dried after bottle sterilization. These 66 samples, however, were collected in a large sterile bottle and held therein for several minutes before transfer to a regular bottle for dechlorination. The holding time in the first bottle was determined by subtracting from 20 min. the theoretical displacement time of the contact tank for the sewage flow recorded at the time of collection. All samples for bacteriological analysis were planted in lactose broth within 5 min. following dechlorination. Three portions of four dilutions each were subjected to the confirmed test using brilliant-green bile in accordance with "Standard Methods." Calculations were made for M.P.N. per 100 ml. on the basis of both presumptive and confirmed tests. The second sample of each pair, collected simultaneously in plain sterile bottles, was tested for residual chlorine by the ortho-tolodine-arsenite method prescribed by "Standard Methods" for water analysis. Total, free, and combined chlorine were recorded.

For purposes of illustrating the effect of longer contact time in the second stage of chlorination, those samples held in a bottle before dechlorination, together with the corresponding samples of settling tank effluent, are referred to hereinafter as Series II; all others are grouped under Series I.

Detention Time—Chlorine Contact Tank

The time of flow through the chlorine contact tank, determined by the time of passage of vegetable and fluorescein dyes, is shown in Figure 5. Dyes were added to the settling tank effluent at the point of chlorine application. At this point considerable turbulence was induced by the discharge from the settling tank effluent channels. The contact tank is about 60 ft. long, 7 ft. wide, and 9 ft. deep.

The time interval from addition of the dye and its first appearance at the

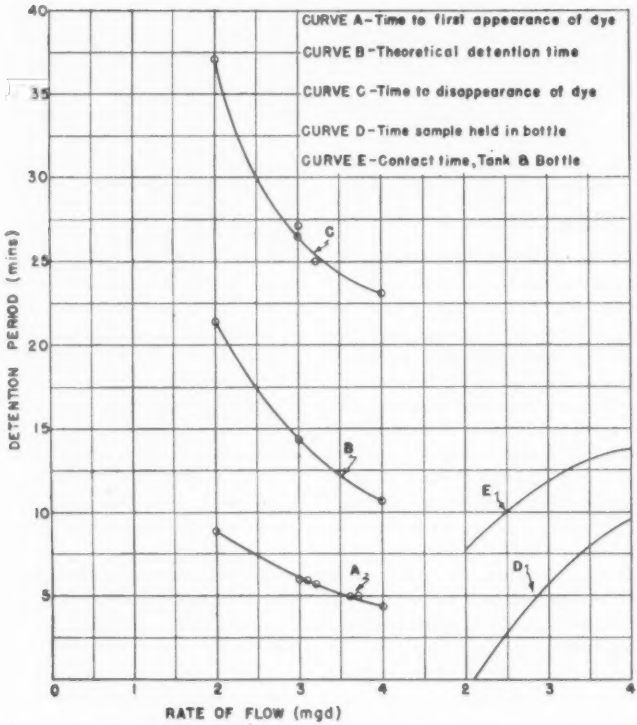

FIGURE 5.—Time of flow in chlorine contact tank, Traverse City, Mich.

point of discharge is plotted in curve A. Time from introduction to last traces at the discharge point is plotted in curve C. The period of contact provided by the tank is thus shown to lie at varying points between curves A and C. The theoretical time of passage, based on displacement, is plotted in curve B. The time interval between 20 min. and curve B is plotted in curve D, representing the time samples were held in the bottle for Series II. The minimum time of contact of chlorine and sewage is the sum of the ordinates of curves A and D, plotted as curve E. Generally the samples were held for 5 to 8 min. in the bottle following a minimum contact period of 5 to 7 min. in the tank.

Fluctuations in Sewage Flow and Chlorine Requirements

All sewage was pumped to the treatment plant by two pumping stations. The main pumping station was equipped with three float-controlled, electrically-operated sewage pumps having a maximum combined capacity of 4 m.g.d. The other station had a capacity of about 0.4 m.g.d. Rates of flow at the plant varied between 1.0 and 4.2 m.g.d. during the 24-hr. period on the days of the survey. An indicating, recording, and totalizing mechanism actuated by a Venturi tube on the plant influent line records all flows.

In order to demonstrate typical flow

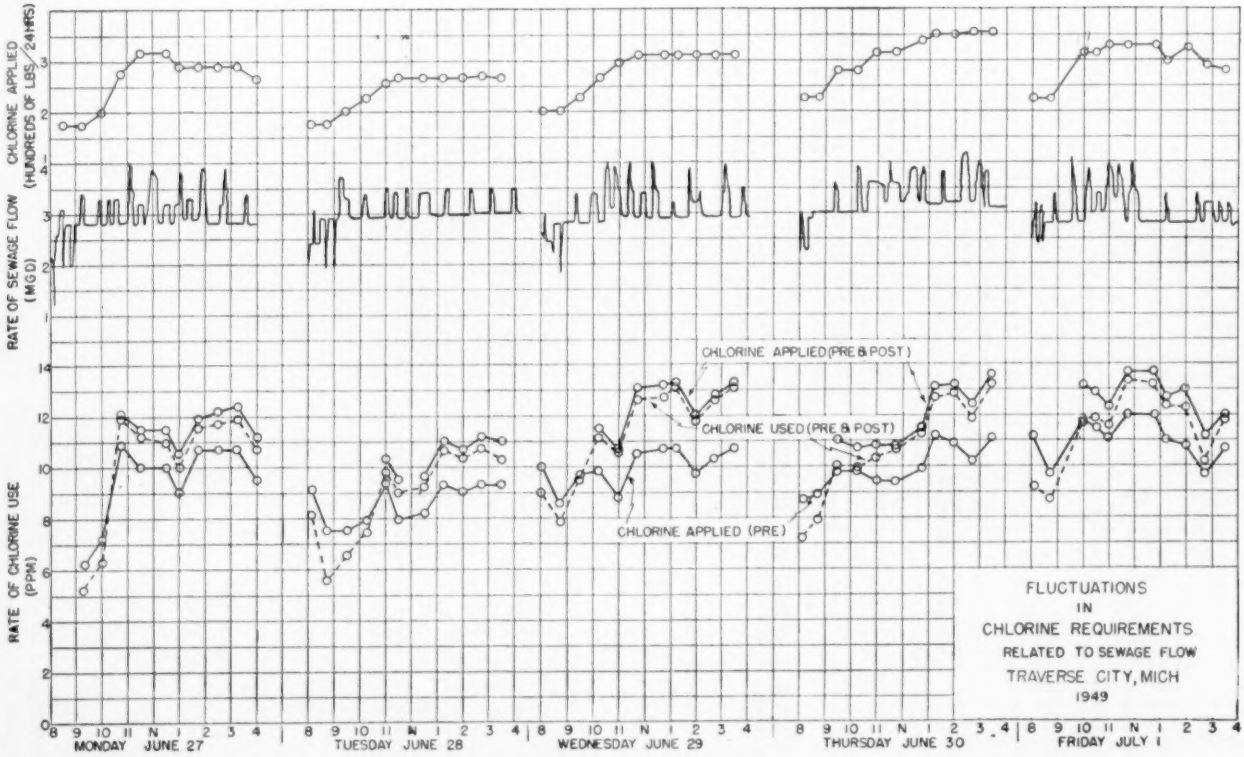
RATE OF SEWAGE FLOW (MGD)
CHLORINE APPLIED (HUNDREDS OF LBS./24 HRS)
RATE OF CHLORINE USE (PPM)
CHLORINE APPLIED (PRE & POST)
CHLORINE USED (PRE & POST)
CHLORINE APPLIED (PRE)
FLUCTUATIONS IN CHLORINE REQUIREMENTS RELATED TO SEWAGE FLOW TRAVERSE CITY, MICH 1949
MONDAY JUNE 27
TUESDAY JUNE 28
WEDNESDAY JUNE 29
THURSDAY JUNE 30
FRIDAY JULY 1

FIGURE 6.—Fluctuations in chlorine requirements related to sewage flow, Traverse City, Mich., June 27–July 1, 1949.

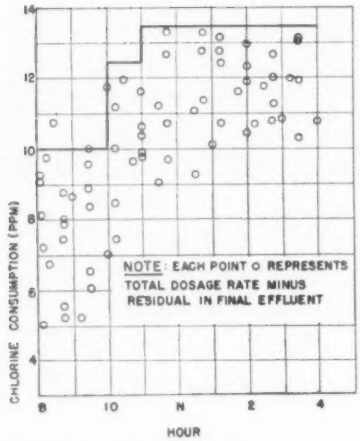

FIGURE 7.—Total rates of chlorine consumption representative of survey period.

patterns occurring during the summer season prior to cherry canning operations, the flow charts for the sampling periods of several days are plotted in Figure 6. Flow rates during the cherry canning season were slightly more uniform and generally of greater magnitude. Although these fluctuations in the flow pattern are much more abrupt and of much greater magnitude than would be expected in a gravity flow system, the peaks are generally of short duration compared with the detention period of the settling tanks. It may reasonably be assumed that some leveling off of these peak rates occurred in these tanks, materially reducing problems of control of chlorine dosage.

It was attempted throughout the survey to control the rate of chlorine feed at the two points of application in such a manner as to produce consistently a residual not to exceed 0.5 p.p.m. in the settling tank effluent and as close as practicable to 0.5 p.p.m. in the contact tank (final) effluent. Rates of feed were adjusted on the basis of predicted patterns of chlorine demand based on experience, on the last previous observation of chlorine residual, and on the rate of sewage flow. It was sometimes necessary to interpret all three factors in order to arrive at the proper rate.

Chlorine dosage, both in terms of pounds per 24 hr. and in relation to sewage flow (p.p.m.), is plotted in Figure 6. The rate in terms of sewage flow was based upon the reading of the sewage Venturi meter at the time of sampling. These points are connected without regard to intermediate flow rates. Here, as in the survey of the previous year, some rough diurnal trends may be observed. However, neither the precise time of occurrence of such changes nor their magnitude was predictable.

Rates of chlorine applied for pre-chlorination and total rate of application for the two stages are plotted in Figure 6. The difference in the two ordinates at any point represents the rate of post-chlorination, or the rate applied to the settling tank effluent. It may be noted that, generally, post-chlorination was not started until after 10:00 AM. Usually chlorine residuals in the settling tank effluent were sufficiently high during the early morning hours to produce a residual of 0.5 p.p.m. or more in the contact tank effluent without further addition. The magnitude of this residual for all sampling periods plotted in Figure 6 may be determined by subtracting the ordinate of the broken line from that of the top solid line.

It is indicated in Figure 6 that a chlorine residual was observed in the final effluent for all samples. As discussed later, this was true during the entire survey. It is shown later (Tables VII and VIII), however, that chlorine residuals were not always observed in the settling tank effluent, confirming the observation of the previous year's study on single-stage pre-chlorination at the same plant and at Charlevoix. This ability to consistently maintain chlorine residuals in the final effluent

TABLE VI.—Range in Rate of Chlorine Feed and Number of Adjustments in Rate Required for Residual Chlorine Control

Date (1949)	Pre-Chlorination			Post-Chlorination		
	Range of Rate		No. Adjustments in Rate	Range of Rate		No. Adjustments in Rate
	(lb./24 hr.)	(p.p.m.)		(lb./24 hr.)	(p.p.m.)	
June 23	200–225	6.8–11.1	4	50–75	1.5–3.7	4
24	0–225	5.1–10.0	3	30–150[1]	1.1–6.0	5
27	175–275	6.2–11.7	5	0–40	0.0–1.7	2
28	175–225	7.5–9.3	2	0–45	0.0–1.9	4
29	200–250	8.6–10.7	2	0–60	0.0–2.6	3
30	225–290	8.7–11.2	3	0–65	0.0–2.5	5
July 1	225–290	9.7–12.0	5	0–50	0.0–2.2	4
5	225[2]	8.4–11.7	0	0	0.0	0
6	0–300	0–13.3	3	0–100[1]	0.0–4.5	2
7	225–300	8.7–12.8	3	0–45	0.0–1.9	3
8	225–300	9.6–12.4	2	0–40	0.0–1.7	2
11	200–300	8.6–12.4	3	0–60	0.0–2.5	5
12	250–350	8.8–14.0	2	0–80	0.0–3.2	4
13	275–375	10.7–13.1	4	0–50	0.0–2.0	1
14	275–400	8.3–14.5	6	0–100	0.0–4.1	3
15	200–435	10.0–16.0	7	0–90	0.0–2.7	6
18	300–390	12.0–16.7	6	0–40	0.0–2.0	4

[1] Rates increased for short period to compensate for failure to pre-chlorinate due to depleted chlorine supply.

[2] Observations from 8:15 to 9:30 AM only.

is of tremendous significance in the control of coliform densities.

How difficult is it to maintain this degree of precision of control? How much overdosage is required? The answers to these questions of great practical interest are indicated in Table VI and Figure 7.

The range in rate of chlorine feed and the number of adjustments in such rates required for this precision of control are tabulated for the entire period of the survey in Table VI. In spite of wide variations in dosage rates, as indicated both in Table VI and in Figure 6, generally not more than 4 adjustments of each chlorinator were required. Referring to Figure 7, it may be noted that total rates of chlorine consumption are plotted for the period June 24–July 6. Such rates are representative for the entire period of the survey. Here the total rate of chlorine dosage for both stages, minus the residual in the final effluent, is represented by a single point for each sampling period. The band occupied by these points has a width of about 3 to 4 p.p.m. It may be observed that if a line were drawn above all points so as to produce a residual of about 0.5 p.p.m. it would be located at about 10 p.p.m. from 8:00 AM to 10:00 AM, 12.5 p.p.m. from 10:00 AM to 11:00 AM, and at 13.5 p.p.m. from 11:00 AM to 4:00 PM. It is interesting to note that, considering all samples for the period, the average rate of overdosage by such a program would not have exceeded 2 p.p.m., whereas instantaneous rates of overdosage would have been generally less than 4 p.p.m. Generally, 2 p.p.m. would have been sufficient for the second stage to produce a residual of about 0.5 p.p.m. Thus, it is indicated that great precision of control of chlorine dosage to consistently produce a residual in the effluent was accomplished with but few adjustments of a manually-operated chlorinator with infre-

quent overdosage. Chlorination equipment automatically proportioned to flow would accomplish the same objective with 3 settings for one chlorinator and a constant rate for the other with relatively small overdosage.

The effectiveness of the control of chlorine dosage with respect to the proposed residual in the settling tank effluent and final effluents is shown in Tables VII and VIII. Of all of the samples of settling tank effluent, 83 per cent contained a residual and 28 per cent contained more than 0.5 p.p.m. Of all of the samples of final effluent, 100 per cent contained a residual, 50 per cent had residuals ranging from 0.35 to 0.75 p.p.m., 31 per cent contained more than 0.75 p.p.m., and 19 per cent contained less than 0.3 p.p.m.

Coliform Density

The coliform densities of all samples of settling tank effluent are grouped in

TABLE VII.—Relationship of Coliform Density and Chlorine Residual in Settling Tank Effluent with Chlorine Applied to Settling Tank Influent; Traverse City, Mich., 1949

Residual Chlorine (p.p.m.)	Per Cent of Samples Not Exceeding Coliform Densities (MPN per 100 ml.) Specified								Total No. Samples
	36	100	500	1,000	5,000	10,000	110,000	Indet.[1]	
(*a*) All Samples (Series I and II)									
0	0	0	13.8	17.2	34.4	44.8	65.5	100.0	29
Tr.–0.1	0	5.0	21.7	26.7	63.5	71.7	96.7	100.0	60
0.15–0.35	8.3	8.3	64.0	69.5	80.6	94.4	100.0	100.0	36
0.5 –0.75	13.3	26.6	66.7	80.0	80.0	80.0	100.0	100.0	15
1.0 –1.5	57.1	71.4	93.0	93.0	93.0	93.0	100.0	100.0	14
1.75–2.0	45.0	60.0	90.0	100.0	100.0	100.0	100.0	100.0	20
Total No. samples[2]	27	35	82	92	131	139	162	174	174
(*b*) Sample Series I									
0	0	0	10.9	10.9	17.6	29.4	47.2	100.0	17
Tr.–0.1	0	3.2	19.3	22.5	54.7	70.8	93.5	100.0	31
0.15–0.35	11.1	11.1	55.5	63.0	92.5	96.5	100.0	100.0	27
0.4 –0.75	14.3	28.6	85.8	85.8	100.0	100.0	100.0	100.0	7
1.0 –1.5	69.2	69.2	92.5	92.5	100.0	100.0	100.0	100.0	13
1.75–2.0	69.2	77.0	100.0	100.0	100.0	100.0	100.0	100.0	13
Total No. samples[2]	22	25	54	57	78	86	97	108	108
(*c*) Sample Series II									
0	0	0	16.7	25.0	58.3	58.3	91.7	100.0	12
Tr.–0.1	0	6.9	24.2	34.5	72.5	72.5	100.0	—	29
0.15–0.35	22.2	22.2	88.8	88.8	100.0	—	—	—	9
0.4 –0.75	12.5	25.0	50.0	75.0	100.0	—	—	—	8
1.0 –1.5	100.0	—	—	—	—	—	—	—	1
1.75–2.0	14.3	43.0	85.8	100.0	—	—	—	—	7
Total No. samples[2]	5	10	28	35	53	53	65	66	66

[1] MPN values indeterminate; 110,000 or greater.
[2] Number not exceeding coliform densities specified.

TABLE VIII.—Comparison of Coliform Densities Pre-Chlorination *vs.* Pre- and Post-Chlorination

Coliform Density (MPN/100 ml.)	Per Cent of Samples Not Exceeding Coliform Densities Specified			
	Series I		Series II	
	Settling Tank Effluent (Pre-Chlorination)	Contact Tank Effluent (Pre- and Post-Chlorination)	Settling Tank Effluent (Pre-Chlorination)	Contact Tank Effluent (Pre- and Post-Chlorination)
36	20.3	44.4	7.6	83.3
100	23.2	59.2	15.2	93.9
500	50.0	83.5	42.5	98.5
1,000	61.5	90.7	53.1	98.5
5,000	72.0	96.3	80.3	100.0
10,000	79.5	97.2	80.3	100.0
110,000	90.0	100.0	98.5	100.0
Indet.[1]	100.0	100.0	100.0	100.0

[1] Indeterminate, greater than 110,000.

Table VII(*a*) by varying levels of chlorine residuals, showing the percentage of samples not exceeding specified coliform densities. In order to study the effect of contact time in the second stage of disinfection, samples of Series I and Series II are tabulated separately (Tables VII(*b*) and VII(*c*)) for both settling tank effluent and final effluent.

Several observations may be made from these data. It may be noted that for any standard of coliform density, the percentage of samples of settling tank effluent meeting the standard increases with increasing chlorine residuals. Examination of the samples with respect to a standard of 500 MPN per 100 ml. shows that in Series I (Table VII(*b*)) 54 of the 108 samples do not exceed this value. Only 19.3 per cent of the samples in Series I with residuals below 0.1 p.p.m. fall in this group. This proportion increases to 55.5 per cent for residuals of 0.15 to 0.35 p.p.m., to 85.8 per cent for residuals of 0.4 to 0.75 p.p.m., to 92.5 per cent for residuals of 1.0 to 1.5 p.p.m. All samples having residuals of 1.75 to 2.0 p.p.m. do not exceed this standard. Generally similar results are noted in Series II (Table VII(*c*)), except the unexplained reversal of position of the group in the 0.15- to 0.35-p.p.m. residual bracket with those in the 0.4- to 0.75-p.p.m. group. Following these results to Table VIII, it may be noted that whereas Series I settling tank effluents (prechlorination only) met the standard 50 per cent of the time, this same sewage subjected to further chlorination met the standard 83.5 per cent of the time. It will be recalled that post-chlorination contact periods for this series were generally of the magnitude of 6 min. for some particles of the flow. In Series II (Table VII(*c*)), where minimum contact periods were of the general order of 10 to 14 min., a more striking improvement in coliform density is noted. Here, where 42.5 per cent of the settling tank effluent samples were within the 500 MPN standard, 98.5 per cent of the samples subjected to two-stage disinfection were within this group. The improvement is much more striking when a coliform density of 36 MPN per 100 ml. is considered. In Series II, a total of 5 of the 66 samples of settling tank effluent met this standard. With the addition of about 2 p.p.m. of chlorine and a contact period of about 10 to 15 min. certain, 55 of the samples did not exceed this value. Here again the Series II group showed greater im-

provement than Series I, illustrating the importance of longer contact periods.

Statistical Evaluation of Coliform Density

It is of interest to examine the mass of data for the entire survey period by an accepted statistical method in order to establish the following fundamental characteristics, relationships, and variations of coliform densities:

1. The mean density of all samples for (a) settling tank effluent (prechlorination) and (b) final effluent (two-stage chlorination).
2. The stability of coliform densities over the survey period and the variations in real changes in level of quality, independent of the inherent errors of the test procedures.
3. Comparison of levels of mean coliform density for single-stage with two-stage disinfection.
4. Extent of similarity in mean coliform density in the Series I tests and the Series II tests, both for settling tank effluent and final effluent in order to study the effect of contact time on level of coliform density. Such a study involves, basically, the elimination of similarities or dissimilarities arising purely from chance.
5. Extent of similarity of the results of the presumptive test and confirmed test as a measure of coliform density.

The graphical methods of statistical analyses developed by Velz (9) are employed in the following charts and discussions.

The coliform densities for the entire period of the 1949 Traverse City survey are plotted in Figures 8 and 9. Data for Series I samples are plotted separately from the data for Series II. For the subsequent consideration of the effect of contact time on second-stage disinfection, it is significant to establish whether or not the bacterial qualities of the two series were of a comparable level. The adopted procedure involved the following steps:

1. Elimination of all samples not subjected to pre-chlorination.
2. Arranging all coliform density data in order of ascending magnitude.
3. Assigning a serial number, m, to each of the coliform density values $(1, 2, 3 \ldots n)$.
4. Determining the plotting position of each serial value on the X scale, by computing the percentage values obtained from $m/(n+1)$. The plotting position for each value represents the percentage probability that the coliform density was equal to or less than that observed.
5. Plotting of all data on log probability paper and drawing the line of best fit through the plotted points.

It will be noted that the X' scale is an expression of standard deviation, σ, from the 50 per cent probability line on the X scale or deviation from the mean. The Y' scale is readily recognized as the logarithm of the coliform density values plotted on the Y scale.

The point of intersection of the 50 per cent probability line with the plotted line represents the mean density of the mass of data. That real changes in level of mean density occurred during the survey is demonstrated graphically by comparison with the standard slope for 3 portions, which is much flatter than the slope of the plotted line. For purposes of comparison of this type of data by others, it may be noted that the slope of the plotted lines, σ_{Log}, determined by scaling Y' for a unit of X', is 1.25 for Series I and 0.86 for Series II of the settling tank effluent.

Figure 8 covers all samples of Series I, and Figure 9 all samples of Series II. Settling tank effluents and final effluents are plotted separately. It may be noted that the mean density of the settling tank effluent denoted thus (*) on the chart is 640 for Series I and 1,200 for Series II. To determine the shift in mean density during the survey period, within any desired confidence limit, lines may be projected from

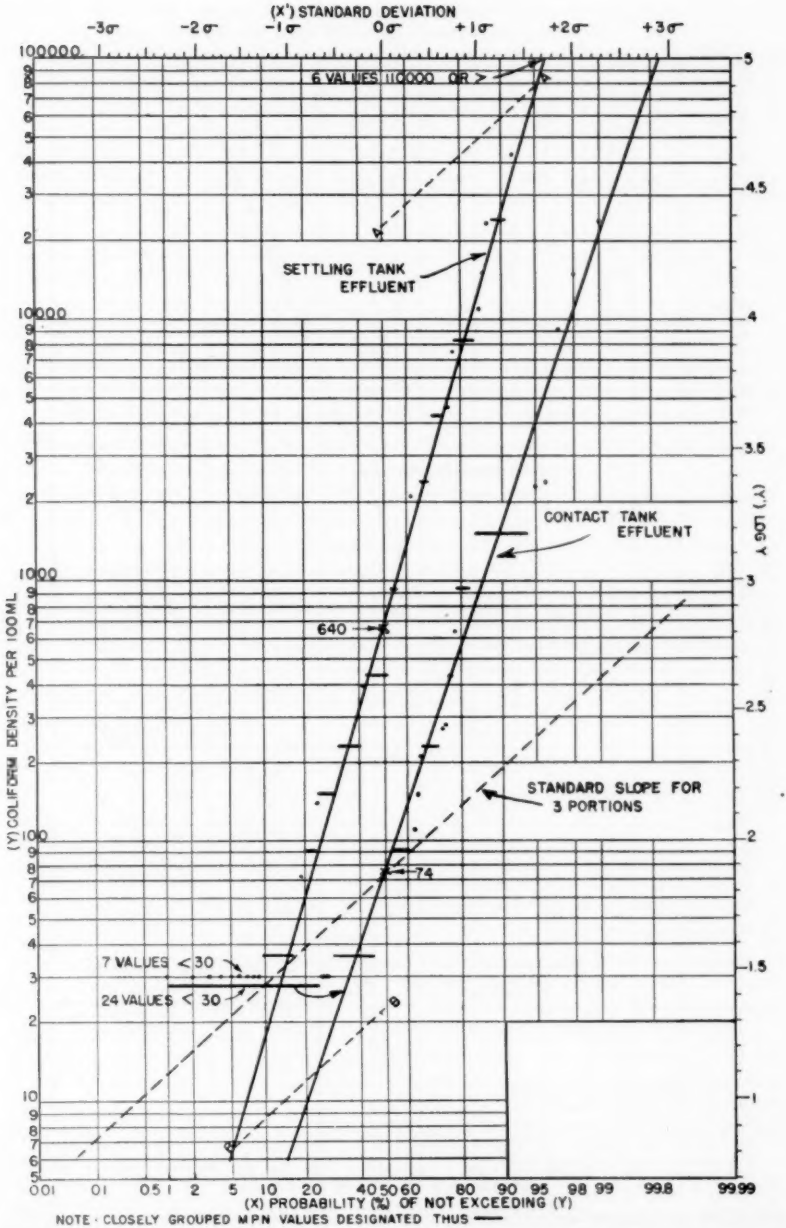

FIGURE 8.—Comparison of coliform densities of settling tank effluent with contact tank effluent, Series I, Traverse City, Mich., 1949.

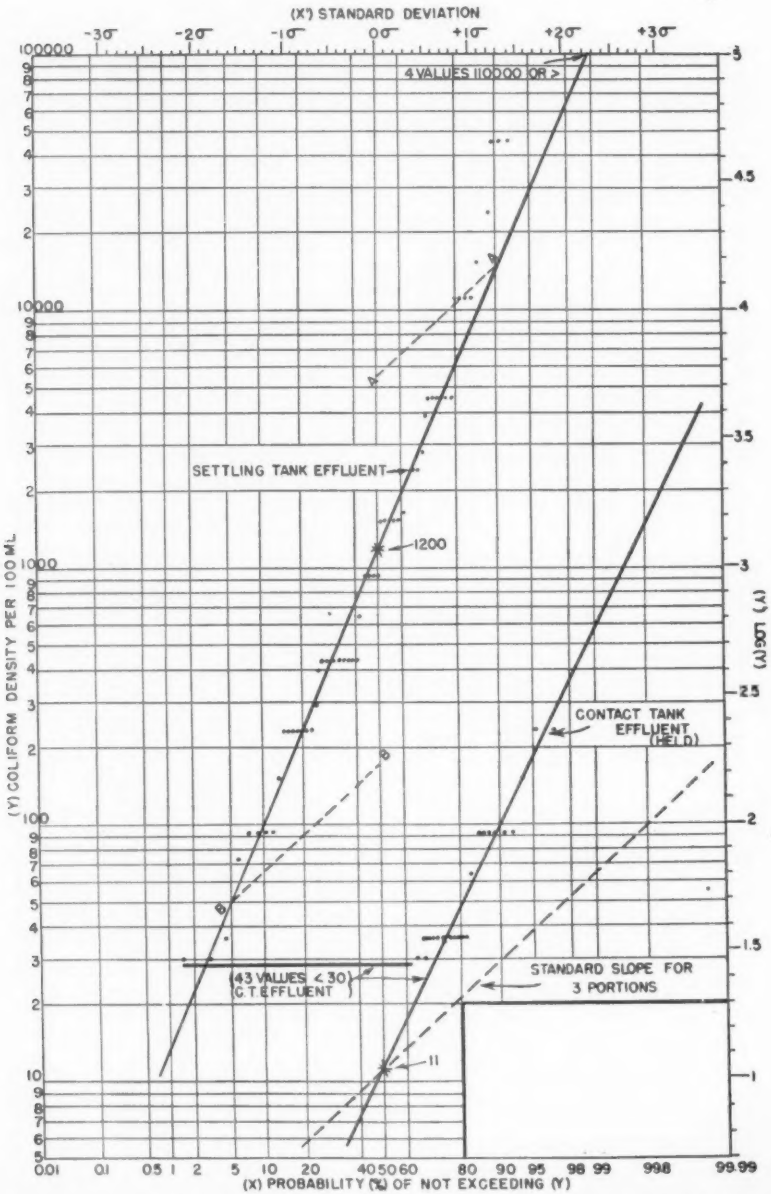

FIGURE 9.—Comparison of coliform densities of settling tank effluent with contact tank effluent (held), Series II, Traverse City, Mich., 1949.

TABLE IX.—Range of Mean Density (MPN per 100 Ml.) Settling Tank Effluent and Final Effluent, Traverse City, Mich., 1949

Source of Samples	No. Samples	Mean Density	Range of Mean Density by Per Cent of Time			
			50%	90%	95%	99%
Series I:						
Settling tank effl.	99	640	160 to 2,700	23 to 23,000	12 to 44,000	4 to 160,000
Final effluent	99	74	25 to 220	5 to 1,100	3 to 2,000	1 to 5,600
Series II:						
Settling tank effl.	66	1,200	530 to 2,700	170 to 6,800	110 to 20,000	55 to 25,000
Final effluent[1]	66	11	<30	<230	<230	<230

[1] After holding in bottle.

points on the plotted line, parallel to the line of standard slope, to the 50 per cent probability line. For a confidence range of 90 per cent, the lower limit of 5 per cent and upper limit of 95 per cent would be examined. The upper limit is determined by starting the parallel line A–A where the plotted line intersects 95 per cent and extending it to the 50 per cent line. Similarly, line B–B determines the lower limit. Thus it may be observed that the 90 per cent confidence range was from about 23 to about 23,000 for the settling tank effluent of Series I. This may be interpreted to mean that 90 per cent of the time the mean density was within the range of 23 to 23,000 MPN per 100 ml. Similar plottings for 50, 95, and 99 per cent confidence ranges provide the results shown in Table IX.

In both Figures 8 and 9 it may be observed that a large number of points for final effluents plotted on the Y axis as 30 are actually less than 30, but of indeterminate density by reason of improper choice of dilutions. This is particularly true of the Series II data, where the majority of the points are less than 30 MPN per 100 ml. It is clear that the position of the plotted distribution line must be estimated on the basis of the few determinate values observed and that the slope of the line may vary considerably from that shown. Any shift of these lines adjusted for this error, however, would not materially change the mean density within practical limits.

The impression gained from examination of the tabulated data in preceding discussions that there is a significant difference between coliform densities of samples obtained from the contact tank directly (Series I) and those held for several additional minutes in a bottle (Series II) is demonstrated here by graphical statistical methods. That a marked difference was found may be noted particularly from Table IX, where the statistical results are tabulated.

It is of interest to note the comparability of coliform density of settling tank effluents of Series I and II. The lines representing distribution of coliform density values for these series are transferred from Figures 8 and 9 to Figure 10. Here graphical methods are employed to determine whether the two series are significantly different, by ruling out chance variations. In comparing these series, it is desired to use a 99.73 per cent confidence level of determination of the means of the two series. This involves the application of ±3 standard errors (SE) of the mean densities. Employing the relationship

$$SE_{\text{mean log}} \equiv \frac{\alpha \log}{\sqrt{n}},$$

$$1\ SE = \frac{1.25}{\sqrt{99}} = \pm 0.126$$

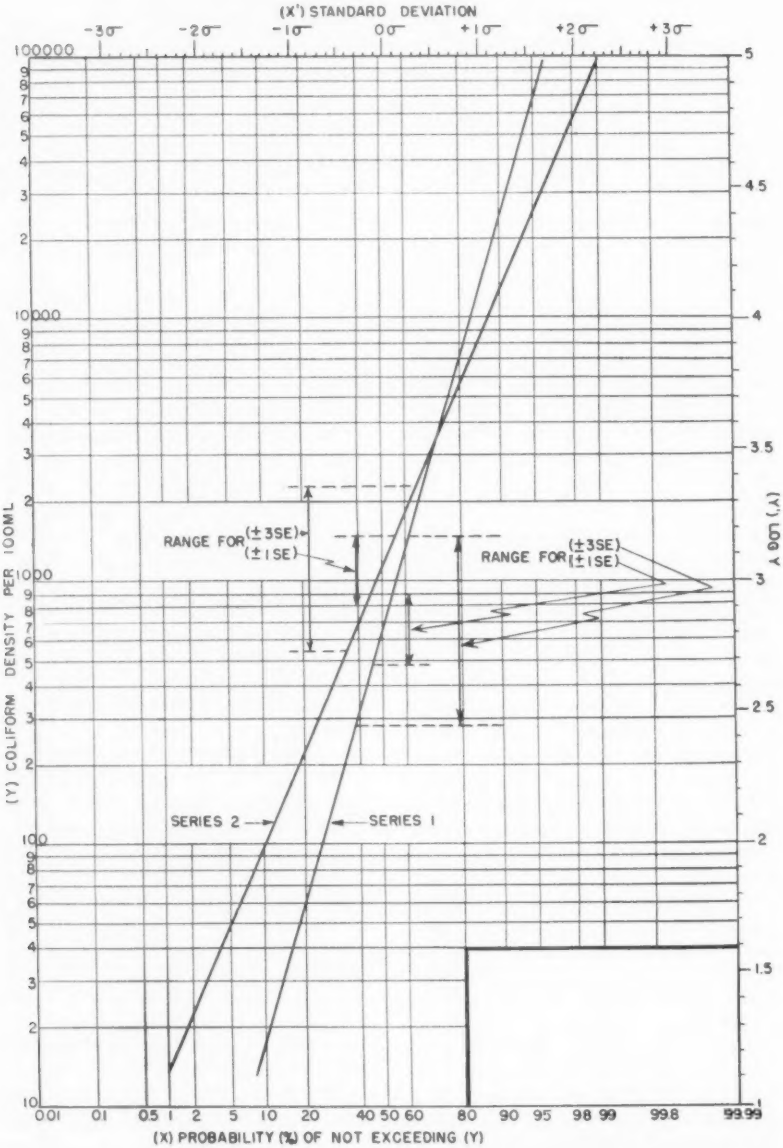

FIGURE 10.—Comparison of coliform densities of Series I with Series II settling tank effluent, Traverse City, Mich., 1949.

for Series I and

$$\text{SE for Series II} = \frac{0.96}{\sqrt{66}} = \pm 0.106.$$

The standard error, measured along the Y′ scale, is plotted above and below the mean density point with intercepts on the plotted density distribution line. Plotting ±1 SE for Series I gives a range of mean density from 480 to 900 MPN per 100 ml. Corresponding densities for Series II are 800 to 1,500. Thus, for 1 SE or within a 68 per cent confidence limit there is overlapping of density values for the two series. When ±3 SE are applied, the range of mean density is about 280 to 1,490 and about 540 to 2,300 for Series I and Series II, respectively. Because there is considerable overlapping of these values, it is reasoned that the difference in mean density of the two series is not statistically significant. This adds considerable strength to the argument that the length of contact time for the second stage of disinfection is a very significant factor in securing uniformly low coliform densities.

It is also of interest to examine the settling tank effluent data on coliform densities to determine whether the results obtained from presumptive tests were significantly different from confirmed test results. The data for these two tests for all samples containing a chlorine residual are plotted in Figure 11. Applying ±3 SE to such data reveals that for this 99.73 per cent confidence limit, the level of mean density by the presumptive test ranges from about 2,800 to 12,800 MPN per 100 ml., whereas the level determined by the confirmed test ranges from about 380 to 1,120. There is a distinct gap between these values, indicating that it is practically certain that the difference is beyond what can be ascribed to chance.

Traverse City Studies—1950

The results of the 1949 studies on two-stage chlorination for disinfection at Traverse City were accepted by the municipal officials as strong indication that an effluent of favorable coliform density could be produced by similar methods employed by regular plant personnel. It was their objective to consistently produce an effluent as free from coliform organisms as possible, within the practicable limits of their resources, before discharge into the river. Because this river flows through the city and discharges into Grand Traverse Bay at a point close to public bathing beaches, they were hopeful of duplicating the 1949 results.

Recognizing the significant reduction in coliform density observed in the previous season by increasing the period of contact for second-stage disinfection, attempts were made to increase the time of contact in the contact tank. By the installation of baffles in this tank the contact time was increased about 50 per cent, thus assuring about 8-min. contact for flows of 3 m.g.d.

Survey Procedures

Routine bacteriological analyses were begun on May 29 and continued through September 28. One sample daily, Monday through Saturday, was collected for the purpose at 3:00 PM, at which time both rate of sewage flow and chlorine requirements were considered to be at about maximum. Chlorine residuals were observed and recorded every hour of the day during the period for control purposes. The sample for bacteriological analysis was collected in a sterile bottle containing sodium thiosulfate. Within a few minutes after collection the dechlorinated sample was planted in lactose broth in four decimal dilutions of three portions each. "Standard Methods" (6) procedures for the confirmed test using brilliant-green bile were employed at all times.

Comparisons with Results of 1949 Studies

Tests for coliform density were performed on 72 samples, the results of which are recorded in Table XI. The

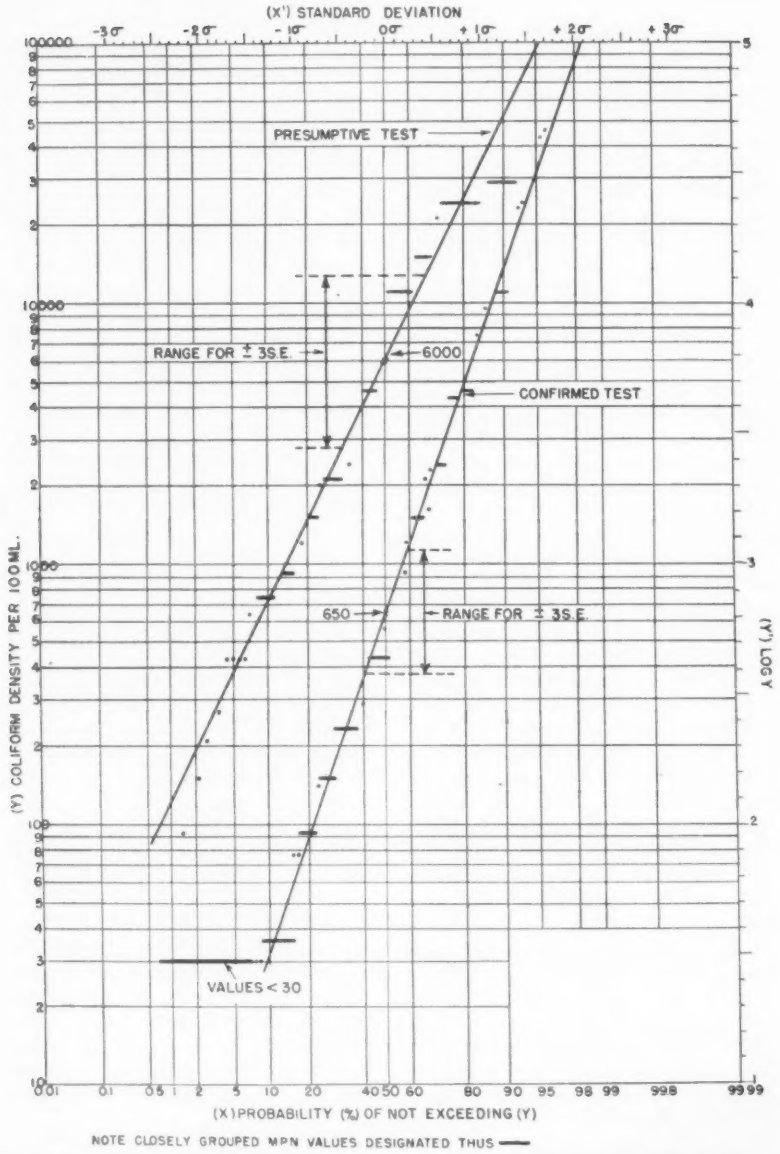

FIGURE 11.—Comparison of coliform densities, presumptive *vs.* confirmed tests, settling tank effluent with pre-chlorination, Traverse City, Mich., 1949.

TABLE X.—Magnitude of Chlorine Residual, 2:00 PM and 3:00 PM Samples, June 7-Sept. 28, 1950; Traverse City, Mich.

Residual Chlorine (p.p.m.)	Settling Tank Effluent				Final Effluent[1]			
	2:00 PM		3:00 PM		2:00 PM		3:00 PM	
	No. Samples	% of Total	No. Samples	% of Total	No. Samples	% of Total	No. Samples	% of Total
<0.05	27	37.5	16	22.2	2	2.8	4	5.5
0.05–0.1	14	19.4	16	22.2	3	4.2	3	4.2
0.15–0.35	16	22.3	23	32.0	14	19.4	7	9.7
0.40–0.75	9	12.5	10	13.9	33	45.8	36	50.0
1.0–	6	8.3	7	9.7	20	27.8	22	30.6
Total	72	100.0	72	100.0	72	100.0	72	100.0

[1] Contact period 6 to 9 min.

results of the observations for chlorine residual are shown in Table X, together with the chlorine residual observed 1 hr. earlier to indicate the amount of chlorine present in the tanks prior to the time of sampling.

Control of Chlorine Residuals.—The same manually-operated chlorinators and methods of adjustment thereof were used as during the survey of the previous year. Rates of sewage flow and chlorine dosage with respect to frequency and range of variation, time of occurrence, and general magnitude were very similar to those observed for the previous year. It is of particular interest to note that the step program of adjustment in rate for a chlorine feed machine proportioned to sewage flow was equally adaptable within the same time intervals and settings to the 1950 results. In both years the same requirement for additional chlorine was noted during the cherry canning season. Relatively the same number of adjustments of the chlorine feeding machine per day were made in both years with generally comparable results.

TABLE XI.—Range of Coliform Density in Final Effluent[1]

Coliform Density (MPN/100 ml.)	Samples not Exceeding Coliform Density Specified.	
	No.	%
36	19	26.4
100	31	43.1
500	52	72.2
1,000	57	79.2
5,000	66	91.7
10,000	69	95.8
110,000	72	100.0

[1] Same samples as in Table X.

It is evident from the data in Table X that control of chlorine dosage to produce a residual in the settling tank effluent was reasonably effective. Generally, the magnitude of the residual at this point was somewhat lower than in the previous year. As no residual at the 2:00 PM or 3:00 PM sampling period was higher than 1.0 p.p.m. and about one-half of these contained less than 0.15 p.p.m., it would appear that dosage rates of about 0.5 p.p.m. higher would have consistently produced residuals within reasonable limits. Data grouped as less than 0.05 p.p.m. include several observations where no residuals were detected. The precise number of such observations is not known, because no differentiation between a trace of chlorine and no residual at all was made by plant operating personnel who recorded these data. However, close examination of the dosage rates ahead of the settling tanks and the additional dosage required to produce a residual in the final effluent

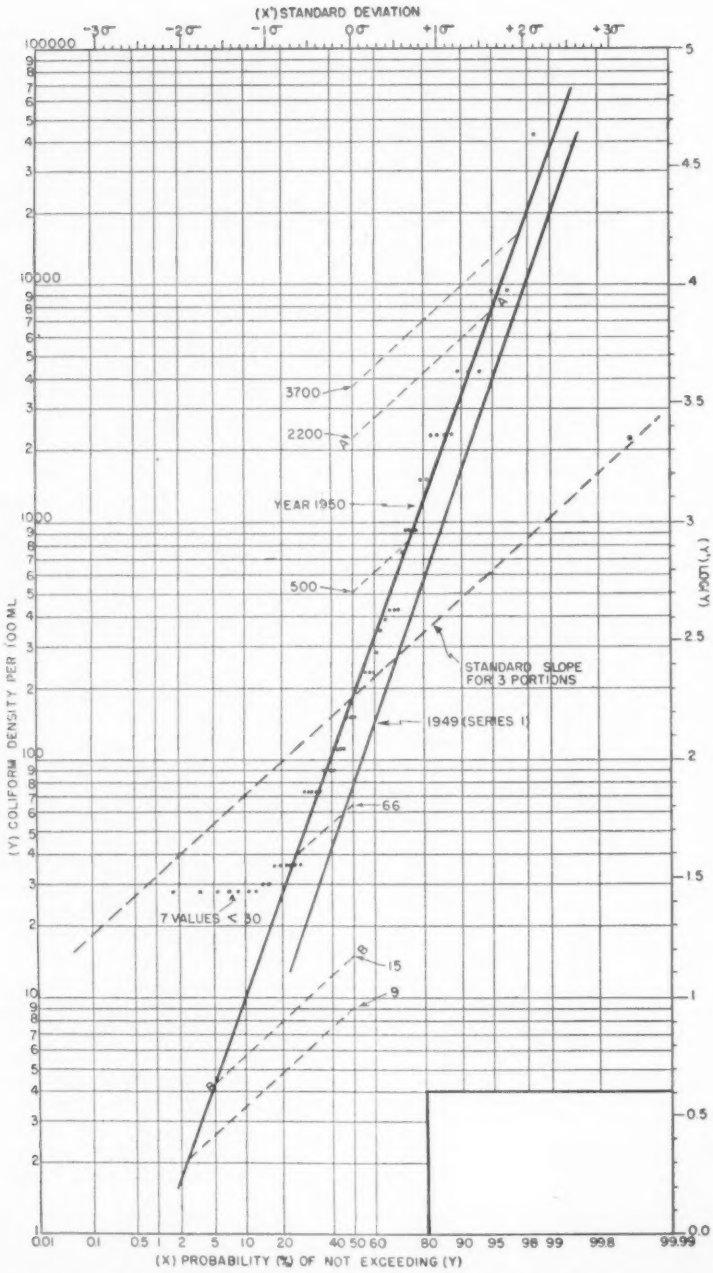

FIGURE 12.—Comparison of coliform densities in final effluent, 1949 (Series I) *vs.* 1950, Traverse City, Mich.

indicates that in all cases very close to a residual in the settling tank effluent was probably produced. It would appear that not more than 1 p.p.m. additional would have been required to consistently maintain a residual of some magnitude less than 2 p.p.m. in the settling tank effluent.

In spite of failure to consistently maintain a chlorine residual in the first stage, all samples of the final effluent contained detectable residuals except when the chlorine supply was nearly depleted or completely exhausted. Only 5 had less than 0.15 p.p.m. and none were higher than 1.0 p.p.m. This remarkable control exercised during all observations of both years with ordinary effort in routine operation practice, on a sewage constantly changing in quantity and in its requirements for chlorine, indicates the marked superiority of two-stage chlorination as here employed for consistent maintenance of a residual in the final effluent, compared with single-stage methods customarily in use today.

Coliform Density.—The distribution of samples of the final effluent by level of coliform density is shown in Table XI. It is of interest to compare these data with those of Series I of the 1949 survey to determine whether there was a statistically significant difference. Such a comparison is made in Figure 12. From Table XI, 16 samples for coliform density, corresponding to the 16 samples of settling tank effluent of Table X for which less than 0.05 p.p.m. of residual chlorine was recorded, are not included in this comparison. However, inclusion of these samples would not have made any statistically significant difference.

The coliform densities of the 56 samples for 1950 are plotted in Figure 12 by methods previously described with respect to Figures 8, 9, 10, and 11. The line designated "1949 Series I" was transferred from Figure 8.

The following observations may be made from the distribution of these values:

1. The distribution of the coliform densities is normal, as evidenced by their adaptability to straight-line plotting.

2. The extent of change in quality during the survey for the two sets of data is very similar, for the lines are nearly parallel.

3. The comparability of the magnitude of coliform densities for the two sets of data is indicated by Table XII. It is apparent that the mean densities of the two sets of data and the shift in mean densities during the period were of the same general magnitude. No statistically significant difference in these data may be noted within limits of practical certainty.

TABLE XII.—Comparison of Test Results for 1949 and 1950

Year	No. Samples	Mean Density[1]	Range of Mean Density[1] by Per Cent of Time		
			50%	90%	95%
1949	99	74	25–220	5–1,100	3–2,000
1950	56	185	66–500	15–2,200	9–3,700

[1] MPN per 100 ml.

On the basis of the close similarity of these data for the two years, and since it was demonstrated in Figure 10 that the levels of coliform density of settling tank effluent of samples in Series I (1949) were not statistically significantly different from samples of Series II of that year, it is reasonable to expect that the extremely low level of coliform density of the final effluent of Series II could have been duplicated by all 72 samples of the 1950 survey if an equally long period of contact of the settling tank effluent with chlorine had been provided.

Holland Studies—1950

The Board of Public Works of Holland, Mich., upon learning of the methods employed at Traverse City to pro-

duce a sewage plant effluent of low coliform density by two-stage chlorination, decided to try similar methods at the Holland municipal plant. Consistent production of an effluent as free as possible of coliform organisms within the practical limitations of the plant facilities was considered essential during the summer season, when the public waters receiving the plant effluent are used rather extensively for recreational pursuits. It was decided that the single-stage post-chlorination method of disinfection as customarily practiced in the plant did not provide sufficiently close control to warrant its continuance if more effective methods could be used practically.

The sewage, almost exclusively of domestic origin from 16,000 to 18,000 people, was conducted to the sewage treatment plant largely by a separate sanitary sewer system. There, after flowing through a bar screen, it was pumped to a battery of Imhoff tanks. Flow rates during the sampling periods generally were within the range of 2.2 to 2.7 m.g.d. Theoretical detention periods of 2.5 to 3 hr. were provided by these tanks, although traces of dye were observed in the effluent within 11 min. after entering the tanks. The settling tank effluent discharged into a small chlorine mixing chamber having a detention period of less than 1 min., thence into a long outfall sewer, submerged throughout most of its length by the lake into which it discharged. This sewer provided a contact period of about 50 min.

Chlorine was applied to the raw sewage prior to pumping and again to the Imhoff tank effluent, by a single solution feed chlorinator with rate of feed proportioned to sewage pumpage rates. Flow to the two points was split and regulated manually by valve adjustment.

Survey Methods

A total of 43 samples for coliform density was collected over a period of 17 days at 10 AM, 1 PM, and 3 PM from August 7–30 from the effluent of the Imhoff tanks. Corresponding samples were collected for chlorine residuals. During approximately these same periods samples were collected also from the outfall sewer at a point approximately 5-min. flow time after the chlorine was added to the settling tank effluent. Samples were obtained also from the outfall sewer at its point of discharge into the lake. It was not possible to collect samples of the final effluent between these points so the coliform density of the effluent after 5-min. contact time could not be determined. All methods of collection and testing were conducted in a manner similar to those employed during the Traverse City studies and in accordance with the "Standard Methods" (6) confirmed test.

Comparison With Traverse City Results

For purposes of comparison the samples should be separated into two groups. The first group, consisting of 25 samples, was collected on the first 10 days of the survey under normal operating conditions. Thereafter, plant operations were seriously disrupted by slow dewatering of all of the contents of one of the Imhoff tanks into the other 3 tanks, thereby greatly increasing the chlorine demand and the amount of suspended solids discharged from the Imhoff tanks. In spite of efforts to control chlorination during this period, only 3 of the 18 samples of Imhoff tank effluent collected during the last 7 days contained any detectable chlorine residual. All except 5 of these 18 samples had coliform densities exceeding 140,000 M.P.N. per 100 ml. It is significant that effluent chlorination reduced the coliform density, after a contact period of about 5 min., as follows: 6 samples to less than 100, 11 under 1,000, and 16 under 5,000 M.P.N. per 100 ml.

Because the first group of 25 samples

TABLE XIII.—Comparison of Coliform Densities in Final Effluent, Two-Stage Chlorination, All Studies

Survey	Year	Minimum Contact Period (min.)	Total No. Samples	% of Samples not Exceeding Effluent Standards Specified[1]					
				36	100	500	1,000	5,000	10,000
Holland	1950	5	25	12.0	36.0	80.0	88.0	96.0	96.0
Traverse City	1950	8–10	72	26.4	43.1	72.2	79.2	91.7	95.8
Trav. City, Series I	1949	5–7	108	44.4	59.2	83.5	90.7	96.3	97.2
Trav. City, Series II	1949	10–14	66	83.3	93.9	98.5	98.5	100.0	

[1] MPN per 100 ml.

was collected under normal conditions, their coliform densities are recorded in Table XIII, together with the results of the two-year study at Traverse City.

The distribution of the Holland samples when plotted by methods discussed earlier in this paper demonstrates that, when compared with Traverse City samples having a minimum actual contact period of 5 to 10 min., the difference in levels of coliform density is not statistically significant within limits of practical certainty. It is readily apparent from Table XIII that there is remarkably close correlation for the three sets of data. It is equally apparent that there was a substantial improvement in effluent quality when the settling tank effluent was in contact with chlorine for a minimum of 10 to 14 min. for all particles of flow. The amount of this difference increases sharply as the quality standard is increased. Although it must be recognized that relatively few samples from the Holland studies are available for comparison for these purposes, and certainly far too few on which to base any definite conclusions, the close similarity of these data with those from Traverse City adds significance to the observations previously discussed.

Summary of Observations and Conclusions

From the studies reported herein the following observations and conclusions may be drawn:

1. Considering all factors, the process of two-stage chlorination is much more dependable than either pre-chlorination or post-chlorination alone, in the consistent production of a coliform density of any relatively low magnitude. It is comparatively much more dependable for meeting effluent quality standards of extremely low order (such as 500 MPN per 100 ml. or lower). Its greatest advantage over either single-stage process is the relative ease with which enduring chlorine residuals can be maintained in the effluent.

2. In all of the tests conducted in all studies of two-stage disinfection it was indicated that coliform densities of 500 MPN per 100 ml. could be produced consistently about 80 per cent of the time with contact periods for post-chlorination of about 5 min., as measured by minimum time of passage of dye. When this time was increased to at least 12 min. it appeared that this standard could be met about 98 per cent of the time. It was further indicated that a standard of 100 MPN per 100 ml. could be met at least 90 per cent of the time and a standard of 36 at least 80 per cent of the time.

3. In all cases, chlorine residuals of varying magnitudes were always present in the final effluent, but sometimes lacking in the settling tank effluent. Control of chlorine dosage to consistently produce a residual in the final effluent, and virtually produce a residual in the settling tank effluent, was accomplished with ordinary effort,

when either manually-operated or automatic rate-controlled chlorine feeding machines were used. Control was exercised by adjustments in dosage rates based on predicted hourly trends both in sewage flow and its chlorine demand and on observation of chlorine residuals at ½-hr. to 1-hr. intervals.

4. Study of hourly patterns of sewage flow and chlorine requirements at one plant indicated that the same three settings of an automatic rate-controlled chlorinator each day during the hours of 8 AM to 4 PM would consistently produce a residual of 0.5 p.p.m. or more in the final effluent with two-stage chlorination, within practical limits of overdosage in small- to medium-sized plants.

5. In all studies of two-stage chlorination generally not more than 4 adjustments of the manually controlled chlorinator were required for the 8 AM to 4 PM period to effectively control chlorine residuals.

6. Rate of chlorine dosage required to produce a residual in two-stage chlorination during the 8 AM to 4 PM period at one plant generally varied from about 8 to 14 p.p.m., with night requirements usually about 5 p.p.m.

7. Generally 2 to 3 p.p.m. of chlorine were required for post-chlorination to satisfy the deficiency in the settling tank effluent so as to consistently produce a residual of some magnitude in the final effluent. At one plant a chlorine residual in the settling tank effluent from a trace to 2.0 p.p.m. was maintained about 83 per cent of the time and 0.15 p.p.m. or more about 50 per cent of the time. All corresponding samples of the effluent following two-stage chlorination contained a residual.

8. The coliform density of the settling tank effluent with pre-chlorination was substantially lowered as the concentration of residual chlorine increased within the limits of 0.05 to 1.0 p.p.m.

9. The importance of time of contact of chlorine with the settling tank effluent was strikingly demonstrated by holding samples of the chlorinated effluent in a bottle for several minutes following an assured contact period of 5 to 7 min. in a chlorine contact tank.

10. Minimum contact time in a long, rectangular, virtually unbaffled chlorine contact tank at one plant, as measured by time of flow of dyes, was about 6 min. when the theoretical displacement time was about 12 to 14 min.

11. The presumptive test for coliform densities seldom yielded the same results as the confirmed test. The extent of the difference in individual values followed no predictable pattern, sometimes being of the same general magnitude, but generally decreasing greatly with confirmation. It is demonstrated by statistical methods that for all samples of settling tank effluent examined at one plant in 1949 the mean density was 650 MPN per 100 ml. for confirmed values, compared with a presumptive mean density of 6,000. It is further demonstrated as a practical certainty that the difference may not be ascribed to chance.

Acknowledgments

The author wishes to express his appreciation and admiration of the thorough and exhaustive field studies and reporting of data by those who actually conducted the surveys herein discussed. To Arthur Corcoran and Ralph Purdy, sanitary engineers, Michigan Water Resources Commission, he is particularly grateful, not only for their management and devoted participation in the field work, but more particularly for their frequent assistance and counsel in the analysis of the data herein reported. The splendid cooperation, interest, and assistance of the superintendents of the four sewage treatment plants studied is also greatly appreciated. For technical advice on statistical evaluation

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AN INVESTIGATION OF THE ROTARY VACUUM FILTER CYCLE AS APPLIED TO SEWAGE SLUDGES *

By Albert H. Halff

Consulting Sanitary Engineer, Dallas, Tex.

Use of the rotary vacuum filter as a means for removing the liquid from sewage sludges has steadily increased. By 1945, 92 municipal sewage treatment plants in this country had installed mechanical processes, mostly vacuum filters, for dewatering sewage sludge (1).

Continuous effort has been made both here and abroad to increase the efficiency of this process of dewatering sewage sludges. Experiments have been conducted by varying the chemical dosage, the speed of rotation, the type of filter cloth, the area of filter, the amount of vacuum, and other factors. Full-scale plants do not lend themselves to experimentation, however, because variation of one factor at a time cannot be assured. In addition, predictions of operating results based on tests using present laboratory techniques have often proven erroneous.

The study reported here was undertaken with the objective of increasing the knowledge of the mechanism of the processes that take place in vacuum filtration of sewage sludges. An improved laboratory method, which would give results consistent with full-scale operating conditions, was a further objective.

History

The author has revised, as follows, Genter's (2) list of the factors in vacuum filtration which bear a mathematical relationship to each other:

1. Effective filter area.
2. Filtration pressure (pressure difference on two sides of the septum).
3. Characteristics of solids (density, particle size, compressibility, etc.)
4. Characteristics of liquid (density, viscosity, etc.)
5. Characteristics of filter septum.
6. Time.
7. Weight of dry solids.
8. Weight of liquid.

Dickey and Dryden (3) and Drew (4) have given a complete review of the filtration equations that have attempted to relate these variables. Two of the most common of these equations—that of Walker, Lewis, McAdams, and Gilliland (5), and that of Ruth (6) were tested, using the data for sewage sludge filtration. Neither of the equations gave good correlations, but that of Ruth was better than that of Walker *et al.* However, neither of these equations was based on data for a filtration process which included the drying cycle, as used in the rotary vacuum filtration of sewage sludges. A search of the literature revealed no measurements of the rate of filtrate removal during the drying portion of this cycle.

The only common factor between the majority of these equations was the acceptance of Poiseuille's (7) and Darcy's (8) laws. Since Dryden's review, Brownell and Katz (9) have suggested a new filtration hypothesis, independent of these laws, but based on Reynolds' number. In order to use

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their method, three characteristics of the solid must be known—particle diameter, porosity, and sphericity. It is not possible to determine these constants for sewage sludges.

It is apparent that a new hypothesis is necessary to correlate the data obtained for the filtration of sewage sludges. The approach followed by Terzaghi (10) in adapting Fourier's method for the solution of heat conduction problems to the consolidation of soils presented a possibility. This method, which has been thoroughly discussed by Terzaghi (11) and Taylor (12), is essentially the approach that has been followed. The proposed hypothesis assumes that the driving force varies with time and distance. In this respect it is different from filtration equations previously developed.

Hypothesis

In order to use the consolidation theory to derive an equation which describes the behavior of the cake on a vacuum filter, it was necessary to assume:

1. A homogeneous slurry.
2. Filtration conducted under constant pressure.
3. Uni-directional flow.
4. Uni-directional sludge compaction.
5. Saturation of the sludge during the process.
6. Incompressible solids and liquids.
7. A negligible quantity of gas.
8. All solids retained on a cloth.
9. Cloth and sludge considered to be an integral unit.
10. Applicability of Darcy's law; also permeability in Darcy's law varies as first power of head loss.
11. If any sludge is subjected to a constant pressure, the ratio

$$\frac{\text{weight of liquid in cake}}{\text{weight of dry solids}}$$

will reach an equilibrium.

12. The portion of the externally applied pressure which is carried by the solids varies inversely as the ratio:

$$\frac{\text{weight of liquid in cake}}{\text{weight of dry solids}}$$

(Figure 1). This is the same as assuming that Hooke's law applies to the solids.

By using these assumptions it can be shown, for a discrete batch of slurry, that the differential equation of the filtration process is

$$c_f \frac{\partial^2 \phi}{\partial z^2} = \frac{\partial \phi}{\partial t} \quad \ldots\ldots\ldots (1)$$

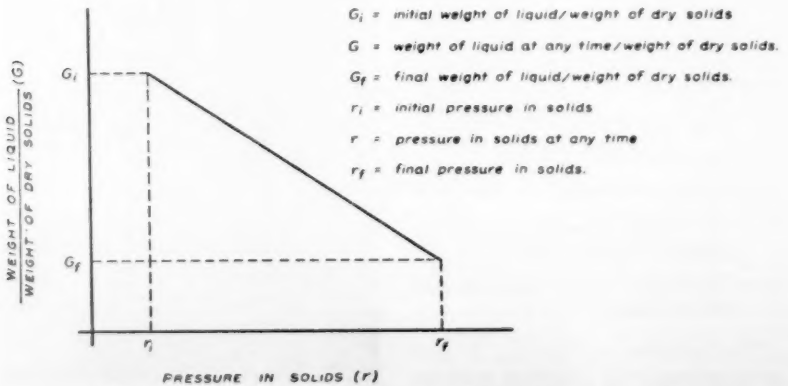
G_i = initial weight of liquid/weight of dry solids
G = weight of liquid at any time/weight of dry solids.
G_f = final weight of liquid/weight of dry solids.
r_i = initial pressure in solids
r = pressure in solids at any time
r_f = final pressure in solids.
WEIGHT OF LIQUID / WEIGHT OF DRY SOLIDS (G)
G_i
G_f
r_i
r_f
PRESSURE IN SOLIDS (r)

FIGURE 1.—Variation of pressure in solids with liquid/solid ratio.

TABLE I.—Relation between Filtration and Time Factor[1]

Filtration Accomplished (%)	Time Factor[2]
0	0
10	0.008
20	0.031
30	0.072
40	0.126
50	0.195
60	0.287
70	0.405
80	0.565
90	0.848
95	1.127

[1] Reference (13).
[2] Dimensionless.

in which

ϕ = a new term called "press" (equal to $K\,p$ in Darcy's law with dimensions $\frac{L\,M}{T^2}$; when Darcy's law is expressed as $v = \frac{K\,p}{L}$);

z = distance above bottom of filter;

t = time;

p = difference in pressure;

K = permeability in Darcy's law with dimensions L^2; and

c_f = coefficient of filtration, further defined as

$$c_f = \frac{c\,\gamma_L}{\mu\,s'\,a} \quad \ldots\ldots\ldots (2)$$

in which

c = constant of proportionality;

γ_L = specific weight of liquid;

μ = viscosity of liquid;

s' = the weight of solids per unit of original volume; and

a = coefficient of compressibility similar to Terzaghi's.

The coefficient of filtration is more easily evaluated by graphical methods, shown later, than by solving Eq. 2.

Integration of Eq. 1 gives

$$F = 1 - \sum_{n=0}^{n=\infty} \frac{8}{(2n+1)^2 \cdot \pi^2} \times e^{-(2n+1)^2 \left(\frac{\pi}{2}\right)^2 T} \quad \ldots\ldots (3)$$

$$j = \frac{\pi}{2}(2n+1) \quad \ldots\ldots\ldots (4)$$

$$F = 1 - \sum_{j=0}^{j=\infty} \frac{2}{j^2} e^{-j^2 T} \quad \ldots\ldots (5)$$

and,

$$T = \frac{c_f\,t}{L_f^2} \quad \ldots\ldots\ldots (6)$$

in which

T = a dimensionless time factor;

L_f = the final thickness of the sludge cake; and

$$F = \frac{\text{accomplished filtration}}{\text{total filtration}} = \frac{\text{volume of filtrate removed}}{\text{volume of filtrate that can be removed at 100 per cent filtration.}}$$

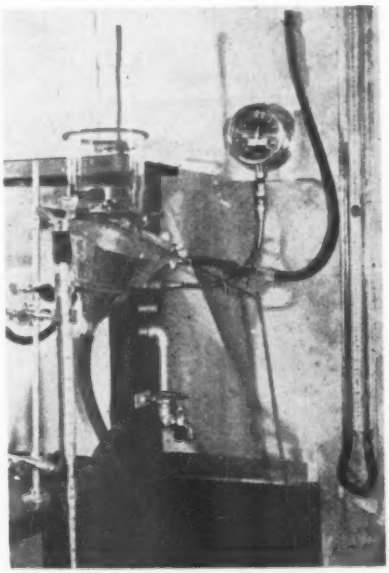

FIGURE 2.—Filter leaf apparatus.

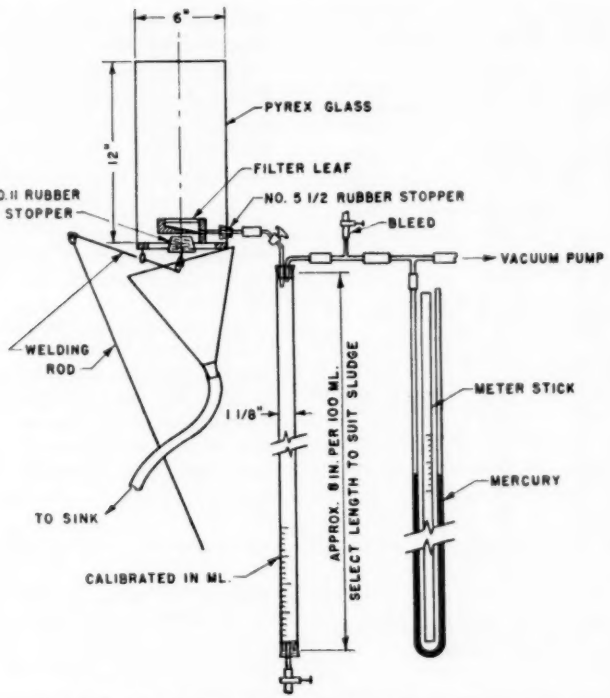

FIGURE 3.—Schematic diagram of filter leaf apparatus.

Eq. 5 represents theoretical relationship between time factor and filtration accomplished. The equation can be solved for the per cent of filtration accomplished to obtain the time factors shown in Table I.

Experimental Procedure

The filter leaf apparatus (Figures 2 and 3) was designed to simulate one revolution of the rotary vacuum filter (14) as closely as could be accomplished in the laboratory. The filter leaf consisted of three parts—a base containing a drain, a ring fitting on the base and to which was soldered a screen to support the cloth, and a retaining ring to hold down the cloth. After the cloth had been attached, the filter leaf was inserted in a battery jar. The drain, which passed through the side of the jar, was connected through a valved line to a calibrated receiving tube. The pyrex jar had a hole in the bottom with a removable stopper so that the slurry could be drained away from the filter.

The procedure for the experiment with sewage sludge was as follows: The sludge was mixed by shaking a large-mouth 5-gal. bottle back and forth 25 times. (The large-mouth bottle was necessary in order to obtain representative samples.) Samples were sized by weight and kept, along with the measured amount of chemical, in the 20° C. constant-temperature room until immediately prior to filtration. The ferric chloride, which had been standardized as recommended by Van Kleeck (15), was measured volumetrically. The temperature of the sludge was

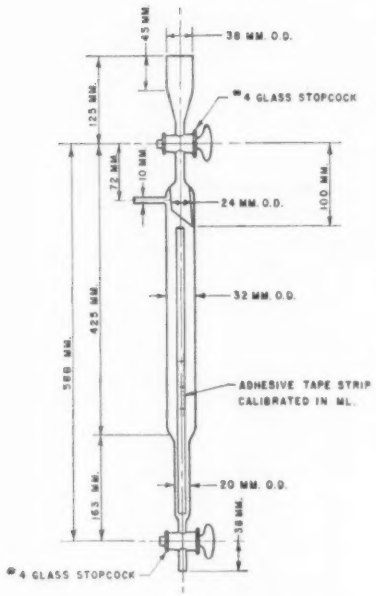

FIGURE 4.—Buchner funnel receiver (not to scale).

maintained constant to within ±0.1° C. of the desired value. When starting a test, the ferric chloride and the sludge were mixed for 1 min. in a 1-gal. Mason jar, by a slow rolling motion. The vacuum was then adjusted, by a bleed, to the desired amount. Next, an inverted Imhoff cone was held over the filter leaf and the sludge was poured into the jar. After pouring, the Imhoff cone was withdrawn. The valve leading to the receiver was opened and a stop watch was started simultaneously. The amount of filtrate entering the receiver was read at specific time intervals. When it was desired to end the submergence of the filter, the plug was removed from the bottom of the sludge container, allowing excess sludge to escape. Readings of time *versus* the quantity of filtrate were continued without interruption until the vacuum broke. The vacuum was considered to have broken when the amount of vacuum had dropped 20 mm. of Hg. This point was not used in any calculations.

After the vacuum had broken, the filter leaf was removed, and the thickness and moisture content of the cake were measured. The measured thickness was used only as a check against the computed thickness. The thickness of the cake was computed using the total weight and specific weights of the sludge and the water, and the area of the filter.

Data to check the equation were also obtained with a Buchner funnel test, using a modified receiving tube (Figure 4). The tube was provided with a tapered section so that the initial volume could be measured accurately. Data from this apparatus should correspond closely to the mathematical case developed.

The procedure for running the Buchner funnel test was the same as that customarily used. However, to check the equation used, different volumes of sludge were poured into the receiver instead of the usual one standard amount.

To perform the experiments it was necessary to measure or control several factors. Values for each factor were selected as follows, except when they were chosen as the variable to be studied:

1. Type of sludge. Experiments were run on well digested, raw, and activated sludges. Sludges were obtained from the Back River sewage treatment plant, Baltimore, Md. (16).

2. Age of sludge. Digested sludge was stored at 20° C. and tested within 10 days of collection. Raw sludge was tested in less than 6 hr. after collection. Return activated sludge was aerated for 1 hr., settled for 12 hr., the supernatant withdrawn, and the sludge aerated for 2.5 hr. before filtration.

3. Bicarbonate alkalinity. The alkalinity as received was comparable with that generally encountered in practice.

4. Solids concentration. The solids

concentration of the digested sludge was 4 to 7 per cent as obtained from the treatment plant. The activated sludge as obtained from the plant was very light. It was settled to obtain a sludge of 0.89 per cent solids. The raw sludge was filtered at 3.9 per cent solids.

5. Type of coagulant. Ferric chloride was used for sludge conditioning.

6. Amount of coagulant. Whenever the chemical dosage was held constant, an amount was used that was representative of actual practice.

7. Filter aids. No filter aids were used.

8. Dissolved gases. The sludge was stirred after the coagulant was added, to eliminate, insofar as possible, all excess gas.

9. Mixing time. In accordance with common practice, a mixing time of 1 min. was selected.

10. Filter area. It is desirable to use a laboratory apparatus large enough that results can be applied to a full-scale plant without error. The area of the filter selected was close to the largest that could be used and still transport a reasonable amount of sludge. One disadvantage of a large area is the increased probability that the filter cloth might contain a defect. In these experiments a new cloth was used for each run. A diameter of 78 mm. (equal to an area of 0.0515 sq. ft.) was selected as a good compromise of the factors mentioned.

11. Underdrainage system. The underdrainage system was designed to provide a minimum loss of head. The screen was selected so that the cloth would provide all of the initial resistance.

12. Filter cloth. There are a great many variables, such as weave, nap, material, weight, and permeability of filter cloths, which need investigation. With present knowledge, it can only be said that the filtration constants obtained are a function of the cloth characteristics. For these tests a long nap, twill-weave wool cloth was selected.

TABLE II.—Effect of Vacuum on Yield of Elutriated Digested Sludge

Exp. No.	Vacuum (lb./sq. in.)	Yield (lb./sq. ft./hr.)
6.8	2.6	3.1
6.4	7.5	3.2
6.3	9.4	3.4
6.6	9.4	3.4
6.2	11.8	3.4
6.5	11.9	3.3
6.7	14.2	3.3

13. Method of fastening cloth. The cloth was fastened on top of the screen by a ring, which was screwed to the base.

14. Amount of vacuum. Young (17) showed that for compressible sludges an increase in vacuum might cause either an increase or decrease in yield, depending on the type of sludge and the thickness of cake. Results of preliminary testing (Table II) indicated that for the over-all process the effect of vacuum was not large for conditioned sewage sludge. The yield per hour was practically constant for pressures greater than 7.5 lb. per sq. in.

Increased flow due to greater vacuum was probably offset by the reduction in flow due to diminished pore diameter, thus producing the result shown. Initially, a high vacuum of near 720 mm. Hg was adopted because it was convenient to use the vacuum pump at this level. It was found that on long filtrations this introduced an error, due to the escape of water vapor into the vacuum line. Later work was done at a vacuum near 500 mm. Hg.

15. Time of submergence. A time of submergence of 1 min. was used because it gave filter yields close to those obtained in actual practice.

16. Time of drying. In these experiments the cake was allowed to dry until the vacuum broke. This time has no analogy in full-scale filtration. Once these data have been taken filter yields may be computed using any desired length of time for drying the

TABLE III.—Effect of Direction of Filtration on Yield

Exp. No.	Liquid Moving	Dry Solids (g.)	Cake Moisture (%)	Time for Vacuum Break	Filtrate Volume (ml.)	Yield (lb./sq. ft./hr.)
14.4	Hor.	7.5	81.0	4′30″	250	4.3
14.5	Up	7.3	81.7	4′	207	4.7
14.6	Down	9.6	80.9	5′40″	252	4.4

sludge. Also, the coefficient of filtration, c_f, may be obtained more accurately when the latter portion of the time *vs.* volume of filtrate data is available.

17. Time of pressure for blow. The blow was applied for 10 sec., or roughly that used in actual practice. In future experiments this could be shortened or the blow eliminated, as it may introduce an error by removing the liquid from the filter cake.

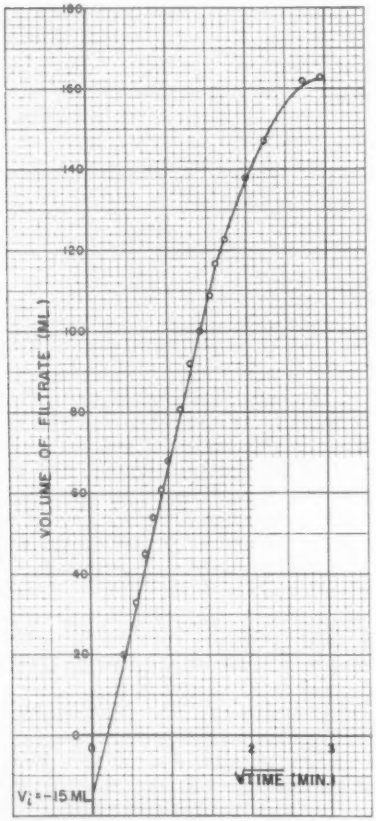

FIGURE 5.—Determination of initial volume of filtrate; Exp. No. 9.1.

18. Amount of pressure for blow. The pressure available was used (approximately 50 lb. per sq. in.).

19. Direction of filtration. The principal flow direction of the filtrate in a full-scale filter during submergence is upward. An experiment was performed to find the magnitude of the effect of direction of filtration.

From the results (Table III) it can be seen that, with the long nap, twill-weave wool cloth, no great difference in the yields will be obtained that are due to direction of filtration alone. This may be a function of the cloth and this test was not made with other cloths. The results show that filtration downward deposited 28 per cent more solids than filtration upward. In interpreting all results, the amount of sludge actually deposited was used; therefore, the effect of this difference was taken into account.

Because the direction of filtration was not a major factor and because the equipment could be made to work much more satisfactorily when filtering downward, this direction was selected.

20. Velocity of vacuum application. In these experiments the full vacuum was always applied immediately.

21. Temperature. The temperature was held at 20° ±0.1° C.

22. Batch dosing or continuous dosing. The apparatus as described is batch dosed; however, the operation

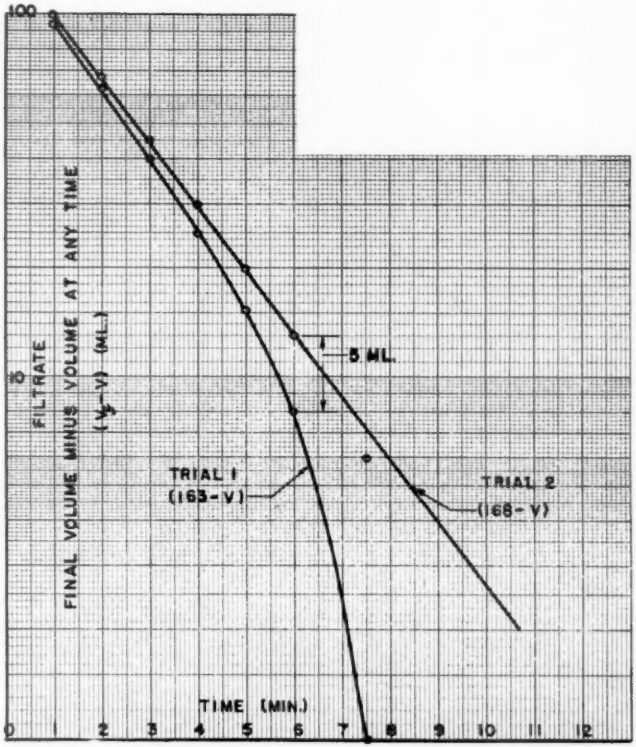

FIGURE 6.—Determination of final volume of filtrate; Exp. No. 9.11.

simulated one cycle of a continuously dosed rotary vacuum filter.

Interpretation of Data

The coefficient of filtration, c_f, can be determined by applying the data from a filtration experiment, as described in the foregoing, by any curve-fitting method which corresponds to the theoretical equations of the proposed hypothesis. Once c_f is known, Eq. 5 or the values of Table I can be utilized to describe the course of filtration.

The following method may be utilized to find the coefficient of filtration. The initial volume of filtrate, V_i, and the final volume of filtrate, V_f, are determined graphically. These volumes do not always coincide with the volume of filtrate as measured.

The initial volume of filtrate is easily determined, because the first section of the theoretical curve is a first-order parabola. Plot square root of time *vs.* volume of filtrate on rectangular coordinate paper (Figure 5). Extend the portion of this curve which fits the data as a straight line to the axis, $t = 0$. The initial volume of filtrate, V_i, is taken as the volume of filtrate read at this intersection.

The final volume of filtrate at 100 per cent filtration, V_f, is determined by the method of Naylor and Doran (18). The volume of filtrate at 100 per cent filtration is estimated as the final volume of filtrate from the data. Using

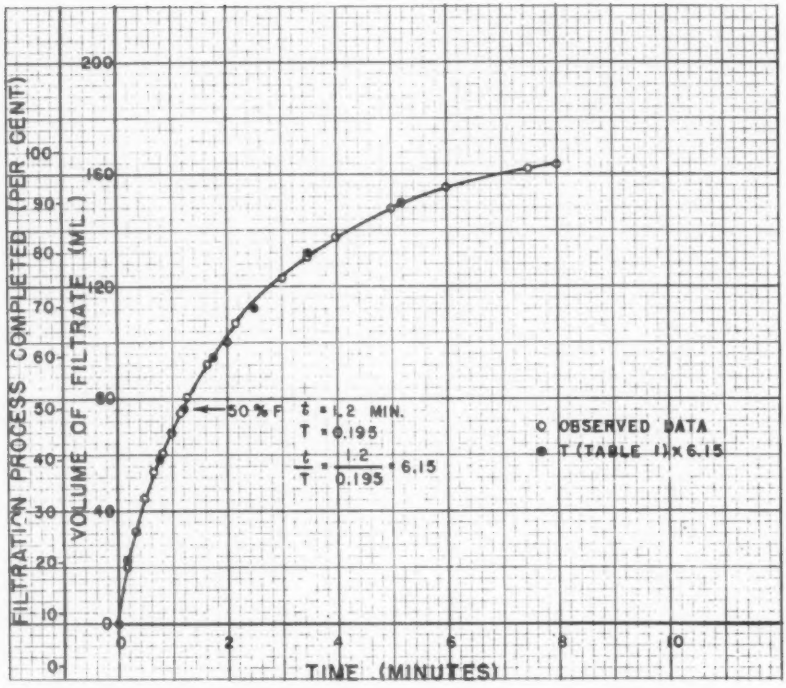

FIGURE 7.—Comparison of observed data with filtration equation; Exp. No. 9.11.

this value and the volume of filtrate *vs.* time data, the volume of filtrate remaining, $V_f - V$, is plotted on logarithmic scale against time on the arithmetic scale (semi-logarithmic paper, Figure 6). If the estimated value of V_f is incorrect, the line will be curved. The line is straightened by graphically applying a correction to the estimated final volume of filtrate, V_f. In plotting the data in this manner, the portion of the data which is under 40 per cent filtration shows as a sharp curve away from the rest of the data in a short space at the beginning of the plot. This portion of the data should be neglected when changing V_f, in order to straighten the line. The final volume of filtrate, V_f, is obtained by correcting the actual final volume of filtrate by an amount necessary to straighten the plot of the volume of filtrate remaining *vs.* time.

The total volume of filtrate is obtained by taking the difference between the initial volume of filtrate, V_i, and the final volume of filtrate, V_f. The per cent filtration accomplished at any time can be computed by dividing the volume of filtrate at that time by the total volume. Each observed volume of filtrate is converted to per cent filtration. A graph (Figure 7) with both volume of filtrate and per cent of filtration plotted against time is a convenient way to show the data.

The coefficient of filtration is computed by substituting a rearranged form of Eq. 6:

$$c_f = \frac{T L_f^2}{t} \quad \ldots\ldots\ldots (6a)$$

Illustrative Example

There follows an example of the computation of the coefficient of filtration by the method just described. Given the following data:

Experiment No.	9.11
Source of sludge	Baltimore dig. elut.
Bicarbonate alkalinity	634 p.p.m.
Specific gravity of sludge	1.03
Specific gravity of solids	1.73
Solids, as filtered	6.76%
Ferric chloride, dry basis	6.43%
Liquid remaining in cake	40.1 ml.
Weight of sludge in cake	12.3 gm.
Area of filter	47.8 sq. cm.

Time		Volume of Filtrate (ml.)	Computed F (%)
Min.	Sec.		
0	0	0	8.2
	10	20	19.1
	20	33	26.2
	30	45	32.8
	40	54	37.7
	50	61	41.5
1	0	68	45.4
	10	75	49.2
	20	81	52.5
	40	92	58.5
2	0	100	62.8
	20	109	67.8
3	0	123	75.5
	30	131	79.8
4	0	138	83.5
5	0	147.5	88.8
6	0	155	92.9
7	30	162	96.7
8	0	163[1]	97.3

[1] Vacuum broke.

The initial volume of filtrate was determined (Figure 5) as -15 ml. The corrected value of the final volume of filtrate was determined (Figure 6) as 168 ml. This is 5 ml. more than had been removed when the vacuum broke (163 ml. from given data). At this time there was 40.1 ml. in cake (from data), therefore this 5 ml. would have come out of the cake had the process gone to completion and there would have been $(40.1 - 5) = 35.1$ ml. in the cake at its final thickness.

The total hypothetical volume $= 168 - (-15) = 183$ ml. The last column in the given data represents per cent filtration for each volume of filtrate computed as follows (for 20 ml. of filtrate):

$$\frac{20 + 15}{183} = 19.1 \text{ per cent filtration}$$

Each 10 per cent filtration is plotted 18.3 ml. apart, starting at 0 per cent filtration equal -15 ml. (Figure 7). After the extra scale has been placed alongside the volume of filtrate scale, the time for 50 per cent filtration is read from the curve.

The final thickness of cake is computed as follows:

Final thickness of liquid

$$= \frac{40.1 - 5.0}{47.8 \times 1.0} = 0.734 \text{ cm.}$$

Final thickness of solids

$$= \frac{12.3}{47.8 \times 1.73} = 0.151 \text{ cm.}$$

Final thickness of cake $= 0.885$ cm.

Compute c_f from Eq. 6a:

$$c_f = \frac{0.195 \times 0.885^2}{1.2} = 0.127 \text{ sq. cm./min.}$$

where T is the value of time factor at the arbitrarily chosen value of 50 per cent filtration (from Table I), t is the value of time at 50 per cent filtration (from Figure 7), and L_f is the final thickness, as computed above. Compute the ratio:

$$\frac{\text{final weight of liquid}}{\text{final weight of solids}} = G_f$$

$$G_f = \frac{35.1 \times 1.0}{12.3} = 2.87$$

For a graphical check of the hypothesis against the data, the actual value of time at 50 per cent filtration is divided by the value of time factor at 50 per cent filtration (0.195). The selection of 50 per cent was arbitrary and any other proportion could have

been used. Each other value of time factor in Table I is then multiplied by the constant thus obtained and the values (constant × time factor) are plotted opposite the corresponding per cent filtration. The curves should coincide if the hypothesis agrees with the data.

One additional set of data must be determined to project the results to a full-scale filtration. The weight of sludge that will be picked up by the filter in any time must be determined by experiment. These data on loading *vs.* time of submergence can be obtained from the filter leaf apparatus, but cannot be obtained from the Buchner funnel test.

Results

Three different methods were used to confirm the hypothesis developed. (a) The theoretical values of filtration *vs.* time factor (Table I) were plotted against the data (volume of filtrate *vs.* time) and the coincidence noted. (b) The sludge was subjected to a constant pressure and at the end of the process the ratio $\frac{\text{weight of liquid}}{\text{weight of solids}}$ was checked for constancy (3). The thickness of the cake was varied and the constancy of c_f in Eq. 6 was checked.

Figure 7 is illustrative of the method of checking actual volume of filtrate *vs.* time against theoretical values of filtration accomplished *vs.* time factor. The method of adding the theoretical values to the graph was described in the previous section. Figure 7 is typical of the curves illustrating this agreement. For purposes of brevity, no further graphs are included. The disagreement where solids concentration in the feed is the variable is illustrated later.

The Buchner funnel test was of primary interest because it most nearly corresponded to the theoretical case developed. An experiment was performed in which the rate of filtrate removed from five different thicknesses of sludge was determined. For all of these data, curves similar to Figure 7 were drawn and the equation described the data consistently.

The final moisture content of the cake was computed by the methods previously described (Figure 6). Using this final weight of liquid, the ratio

$$\frac{\text{final weight of liquid}}{\text{weight of dry solids}} = G$$

was computed, as shown under the heading "Illustrative Example" in the section "Interpretation of Data," for five different thicknesses of digested sludge cake which had been dewatered in the Buchner funnel. The mean value of G was 3.14, with a standard deviation of 0.19.

The third method of checking the proposed hypothesis was the constancy of c_f in Eq. 6. All other factors in this equation are known or can be computed. This constant for five different thicknesses of digested sludge cake was 0.0336 sq. cm. per min. with a standard deviation of 0.00028 sq. cm. per min.

Experiments with a different digested sludge were performed with the filter leaf apparatus. Ten different thicknesses of sludge were obtained by varying the time of submergence. Graphs similar to Figure 7 were plotted and showed good coincidence between actual and theoretical values. The arithmetic mean value of G was computed to be 3.14 and the standard deviation 0.11. The arithmetic mean value of c_f was 0.153 sq. cm. per min. with a standard deviation of 0.016 sq. cm. per min. for 10 thicknesses of cake.

One test was run using the filter leaf apparatus and the same digested sludge as was used to obtain constants with the Buchner funnel test. The value of G for the filter leaf test was 4.2, as compared to 3.1 for the Buchner funnel test. The value of c_f for the filter leaf test was 0.055 sq. cm. per min., compared to 0.034 sq. cm. per min. for the Buchner funnel test. These answers are significantly different.

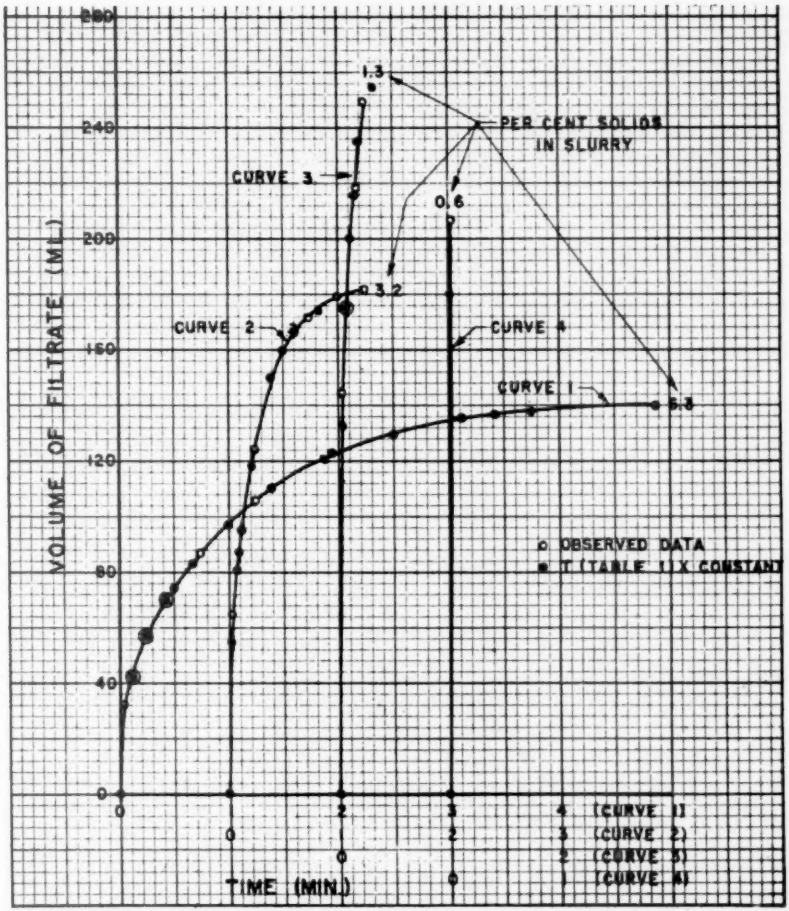

FIGURE 8.—Effect of solids concentration on filtration; Exp. No. 12 (Buchner funnel test).

The filter leaf and Buchner funnel differ in four respects—(a) The liquid removed from the slurry in the Buchner funnel must have been associated with the solids applied to the funnel, whereas liquid removed by the filter leaf apparatus may have come from solids which drained away when the plug was removed; (b) the wire supporting the cloth on the filter leaf assures that cloth is the initial resistance, whereas on the Buchner funnel the small holes in the poreclain base may alter the cloth resistance; (c) the cake is drawn down tightly around the edges of the filter leaf and thus the air must break through the cake, whereas on the smooth-sided Buchner funnel the cake usually cracks at the edge; and (d) weight of solids deposited on the filter leaf adjusts itself to existing conditions, whereas the solids deposited on

the Buchner funnel are equal to those applied.

More study would be necessary to establish which of these differences caused the difference in the constants obtained, but ignorance on this point does not negate the facts established by these experiments; namely, that the hypothesis holds for both types of apparatus.

Another difference in the results obtained by the two types of apparatus was in the weight of sludge, measured and computed. The weight of sludge was computed by applying the total volume of filtrate times the per cent solids as applied to the filter. Whereas the measured and computed sludge weights must, of necessity, check closely on the Buchner funnel test, it was noted in the filter leaf test that the computed weight of sludge was consistently greater than that actually found on the filter leaf. The small amount of sludge on the sides of the filter leaf did not account for this difference. This meant that some liquid was being drawn from the sludge which was not dewatered sufficiently to stick to the filter leaf. The amount by which

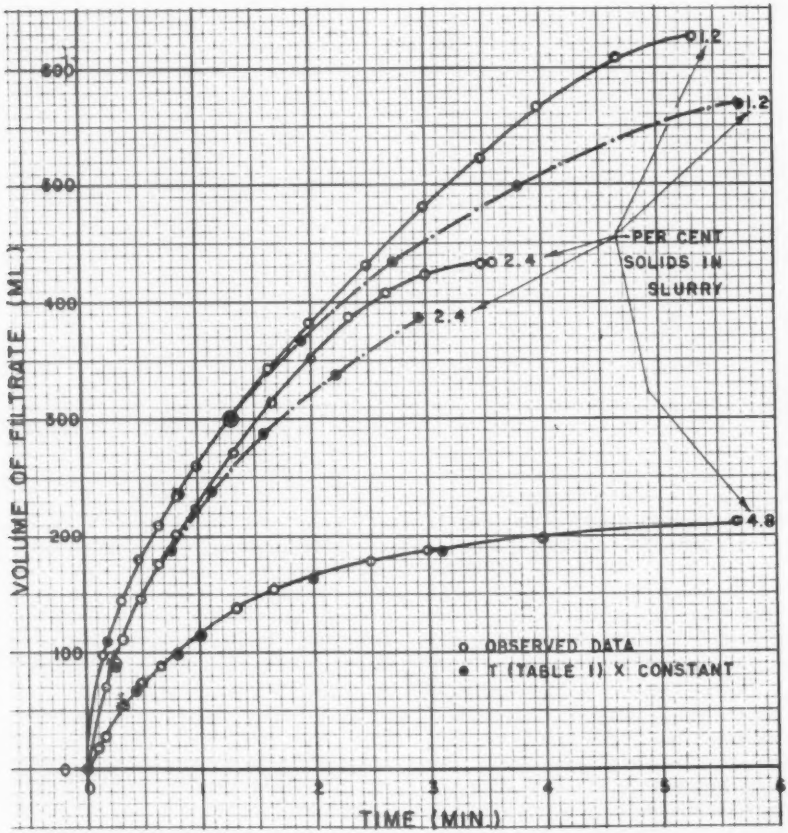

FIGURE 9.—Effect of solids concentration on filtration; Exp. No. 14 (filter leaf apparatus).

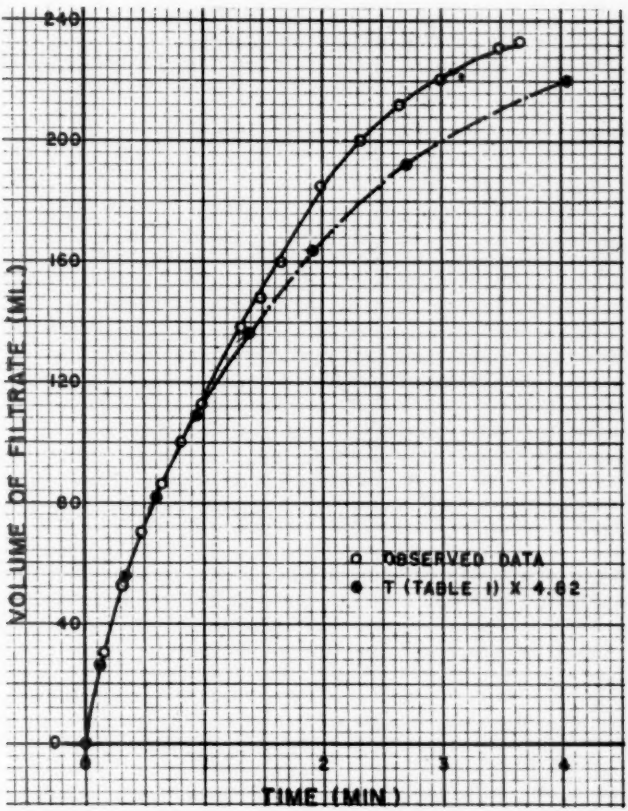

FIGURE 10.—Filtration of raw sludge, 3.8 per cent solids; Exp. No. 15.3.

the weight of sludge was greater than the weight of sludge measured was directly proportional to the time of submergence. Other factors affecting this difference in weight were not studied. This concentration of sludge in the filter tank has been noted in actual practice. Had continuous operation been possible, an equilibrium might have been reached where the measured and computed sludge weights would have checked.

Joseph (19) used the filter leaf apparatus to test different cloths with the same digested sludge. Varying the cloth gave different values of G and c_f, thus indicating that the cloth is an integral part of the sludge filtration process.

Figure 8 shows the effect of varying the solids concentration in the feed when using the Buchner funnel test. The solids concentration was varied by coagulating the sludge and then diluting with different amounts of distilled water. As this changes the characteristics of the slurry, the results can only be interpreted in a general manner. Good agreement between the equation and the data was noted.

The final volume of filtrate could not be computed for digested sludges of 3.3

per cent solids or less, because the vacuum broke before 100 per cent filtration was reached. Thus, G or c_f for slurries of 3.3 per cent or less could not be determined from the Buchner funnel test.

Figure 9 shows the effect of changing the solids concentration in the feed

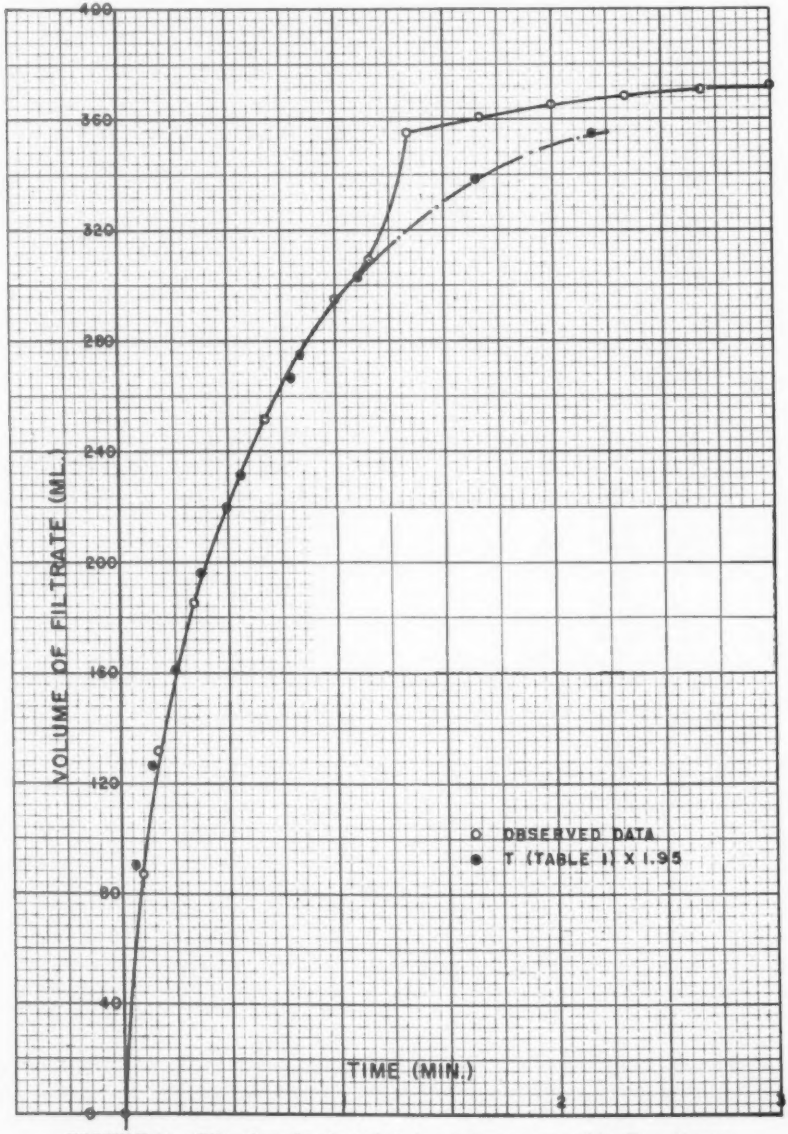

FIGURE 11.—Filtration of activated sludge, 0.88 per cent solids; Exp. No. 16.

when using the filter leaf apparatus. Comparison of the curves with the proposed equations shows good agreement where the solids were 4.8 per cent; correlation was poor below this value. Where deviations occurred the volume of filtrate removed was always greater than that predicted by the equation.

The explanation of this discrepancy may be the same as the explanation given for the difference between the measured and computed weights of solids. Some liquid is drawn from the sludge not retained on the filter. This water is probably being drawn into the filter in all cases, but above approximately 5 per cent solids it can be neglected.

For both kinds of apparatus, then, there was a minimum load which must be applied in order that the proposed hypothesis would apply. This was approximately 5 per cent solids, or a concentration lower than that encountered in actual operation for digested and raw sludges. Activated sludge is usually filtered at a lower solids concentration and although the equation will not give close correlation with this sludge, some of the principles are believed to apply.

Figure 10 shows the results of a test on well-coagulated raw sludge of 3.8 per cent solids. Again, the divergence from fit is noted, which confirms the finding that below approximately 5 per cent solids the equations do not conform.

Figure 11 shows the results of an experiment with activated sludge at 0.88 per cent solids. The thickness was varied by changing the time of submergence. This sludge was very difficult to filter and would not have filtered satisfactorily in commercial practice. The curves show the same divergence from fit as others at low solids concentrations.

The results of these experiments indicate that (a) each individual test will follow the hypotheses as developed, provided the solids in the slurry are above a minimum of approximately 5 per cent; (b) under a constant pressure, the ratio

$$\frac{\text{weight of wet cake}}{\text{weight of dry solids}}, G,$$

reaches a constant if time is infinite; and (c) Eq. 6 holds for digested sewage sludge. Thus, the hypothesis was consistent with laboratory results.

Application

The equation developed (Eq. 5) can be used to solve operating or design

TABLE IV.—Yield from Different Final Cake Thicknesses, Digested Sludge

Sludge		Water Vol.[1,2] (c.c./sq. cm.)	Cake Thickness[2] (cm.)	Water Vol.[1,3] (c.c./sq. cm.)	Cake Thickness[3] (cm.)	Time[4] (min.)	Yield	
Wt. (g./sq. cm.)	Vol.[1] (c.c./sq. cm.)						(g./sq. cm./min.)	(lb./sq. ft./hr.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
0.15	0.071	0.60	0.671	0.471	0.542	1.6	0.094	11.6
0.20	0.095	0.80	0.895	0.628	0.723	2.8	0.071	8.7
0.25	0.119	1.00	1.12	0.785	0.904	4.3	0.058	7.1
0.30	0.143	1.20	1.34	0.940	1.08	6.2	0.048	5.9
0.35	0.167	1.40	1.57	1.10	1.27	8.5	0.041	5.0
0.45	0.214	1.80	2.01	1.41	1.62	13.9	0.032	3.9
0.60	0.286	2.40	2.69	1.88	2.17	25.0	0.024	3.0

[1] Volume per unit area, or thickness in cm.
[2] At 80 per cent moisture.
[3] At 100 per cent filtration.
[4] To reach 80 per cent moisture.

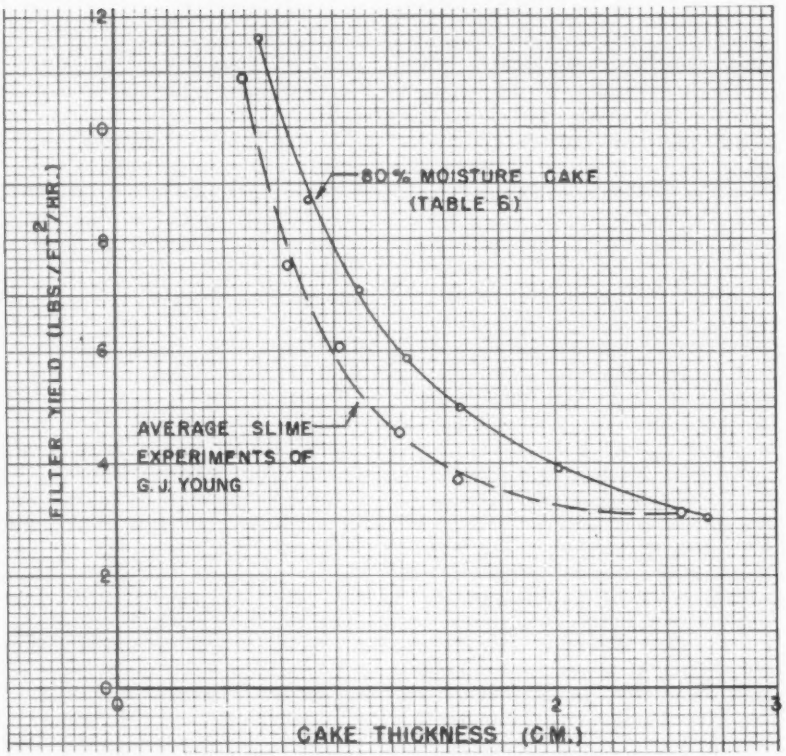

FIGURE 12.—Filter yield *vs.* cake thickness; digested sludge of Exp. No. 8.

problems after the constants have been determined from laboratory tests. Some operating variables, such as (a) leakage in suction system, (b) uneven distribution of inflowing sludge, (c) cake scouring and sloughing, and (d) uneven cake formation, may alter the results of individual runs, but the principles should permit predictions by which better operation can be achieved.

For example, consider the problem: "What is the relationship between yield and cake thickness when operating the filter to obtain a cake of constant moisture content?" To solve this problem, the following data from an experiment on digested sludge are used:

Experiment No.	8
Source of sludge	Baltimore dig. elut.
Bicarbonate alkalinity	354 p.p.m.
Specific gravity of sludge	1.04
Specific gravity of solids	2.1
Solids as filtered	7.4%
Ferric chloride, dry basis	5.9%
c_f	0.158 sq. cm./sec.
G	3.14

Computation of the yields is shown in Table IV and the results are plotted in Figure 12. In this problem the total length of cycle (submergence and drying) is not fixed, and the length of the time of revolution is determined by assuming a weight of slurry and computing the time to bring this slurry to constant moisture content. Cake with 80 per cent moisture was chosen, but any other percentage would give paral-

lel results. This is illustrated by data from experiments by Young (20) on the effect of cake thickness on yield, also plotted on Figure 12.

Method of Computation

Col. 1.—Arbitrarily selected to give results comparable with actual practice.

Col. 2.—Weight of sludge in Col. 1 divided by specific weight of solids (= 2.1).

Col. 3.—Weight of sludge

$$\times \frac{\text{per cent moisture selected}}{100 - \text{per cent moisture selected}};$$

$$\text{Col. } 1 \times \frac{80}{20}.$$

Col. 4.—Volume of solid per unit area plus volume of water per unit area; Col. 2 plus Col. 3.

Col. 5.—Weight of solids

$$\times \frac{\text{final weight of liquid}}{\text{final weight of solids}}, \quad G = 3.14,$$

as determined by experiments on this sludge; Col. 1 × 3.14.

Col. 6.—Final volume of solid per unit area plus final volume of liquid per unit area; Col. 2 plus Col. 5.

Col. 7.—By solving the equation:

$$t = \frac{T L_f^2}{c_f} \qquad (6b)$$

T is found by first solving for

$$F = \frac{\frac{M_i}{s_i} - \frac{M_t}{s_t}}{\frac{M_i}{s_i} - \frac{M_f}{s_f}} \qquad (7)$$

in which F is the filtration accomplished; M_i is the initial moisture content on a weight basis; M_f is the final moisture content on a weight basis; M_t is the moisture content on a weight basis at time, t; s_i is the initial solids content on a weight basis; s_f is the final solids content on a weight basis; and s_t is the solids content on a weight basis at any time, t.

The moisture and solids contents originally applied to the filter are assumed to be the same as those tested in an experiment with this sludge (= 92.6 per cent and 7.4 per cent, respectively). The moisture and solid contents to be obtained are assumed to be 80 per cent and 20 per cent, respectively (Col. 3). The final moisture and solid contents are found from the value of G (3.14 from the experiment) by the following

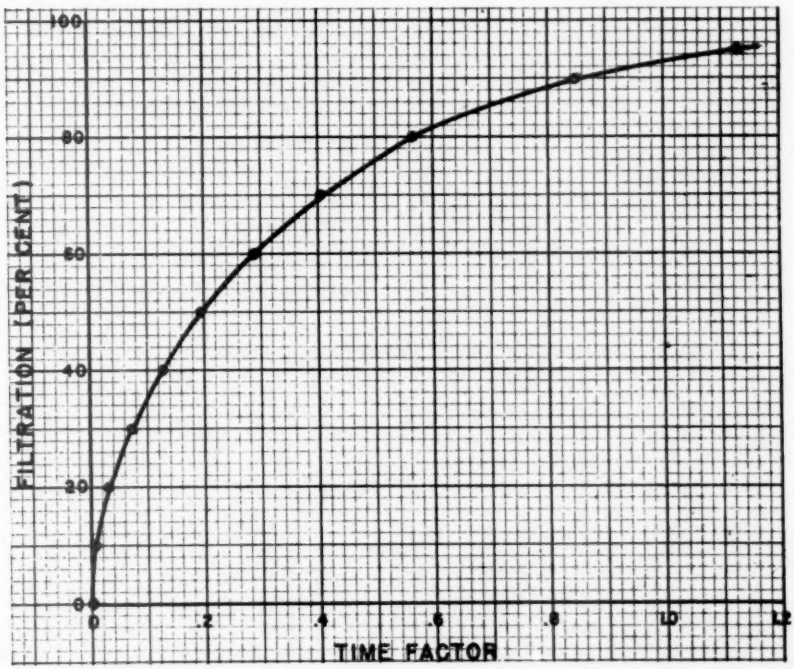

FIGURE 13.—Theoretical per cent filtration completed *vs.* time factor (from Table I).

equation:

$$s_f = \frac{100}{G + 1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

$$s_f = \frac{100}{3.14 + 1} = 24.2 \text{ per cent}$$

$$M_f = 100 - 24.2 = 75.8 \text{ per cent}$$

Substituting in Eq. 7,

$$F = \frac{\frac{92.6}{7.4} - \frac{80}{20}}{\frac{92.6}{7.4} - \frac{75.8}{24.2}} = 0.9$$

The filtration process completed will be the same for all sludges because they are all reduced to the same moisture content. The value of T corresponding to $F = 0.9$ can be read from Table I or from the same values plotted in Figure 13 (= 0.85).

The values of L_f are given in Col. 6; c_f is known from an experiment (= 0.158). Solving Eq. 6b, $t = \frac{0.85\ L_f^2}{0.158} = 5.3\ L_f^2 = 5.3$ (Col. 6)^2.

Col. 8.—Assume no dormant space on the filter. Then the time of revolution will be equal to Col. 7 and yield = Col. 1 ÷ Col. 7.

Col. 9.—Col. 8 × 123.

It may be concluded that the minimum cake thickness will give the maximum yield per hour. Cake thickness is limited by practical considerations. The cake must be at least as thick as the wire winding over the cloth or it will not discharge properly. Cakes that are too thin may be a contributing cause to cloth blinding and also may allow air to be pulled through too soon, thus reducing the vacuum.

Another problem of interest is: "What is the relationship between yield and moisture content for a fixed time of revolution and fixed chemical dose when the time of submergence is varied?"

The solution of this problem shows whether or not the yield can be improved by increasing the speed of the filter while still producing a cake of the same moisture content. The chemical dosage, the moisture content of the slurry, and the time of revolution are assumed to remain constant. The problem is solved for three different times of revolution, but computations are shown for the 5-min. cycle only.

The problem was solved by assuming several times of submergence (Table V). The data from the preceding problem were used, the data necessary to plot Figure 14 being used to establish the load on the filter. The experimental data were obtained from a filter leaf test, as previously described. Once the load has been established, it is a matter of routine computation to determine the moisture contents and yields for different filter cycles.

Method of Computation

Col. 1.—Assumed values to cover the range used in practice.

Col. 2.—Weight of sludge corresponding to times of submergence in Col. 1 (from Figure 14).

Col. 3.—Weight of sludge divided by specific weight = Col. 2 ÷ 2.1.

Col. 4.—Weight of solids × $\frac{\text{weight of liquid}}{\text{weight of solids}}$,

TABLE V.—Computation of Yield for Varying Times of Submergence, 5-Min. Cycle

Submergence Time (min.)	Sludge		Water Vol.[1,2] (c.c./sq. cm.)	Cake Thickness[2] (cm.)	Time Factor	F (%)	M_f (%)	Yield	
	Wt. (g./sq. cm.)	Vol.[1] (c.c./sq. cm.)						(g./sq. cm./min.)	(lb./sq. ft./hr.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
0.25	0.20	0.095	0.628	0.723	1.51	95+	76	0.040	4.9
0.50	0.23	0.110	0.722	0.832	1.14	95	78	0.046	5.7
0.75	0.26	0.124	0.816	0.940	.89	91	80	0.052	6.4
1.0	0.28	0.138	0.879	1.017	.76	88	81	0.056	6.9
1.5	0.32	0.152	1.00	1.15	.60	82	83	0.064	7.9
2.0	0.35	0.167	1.10	1.27	.49	75	84	0.070	8.6

[1] Volume per unit area, or thickness in cm.

[2] At 100 per cent filtration.

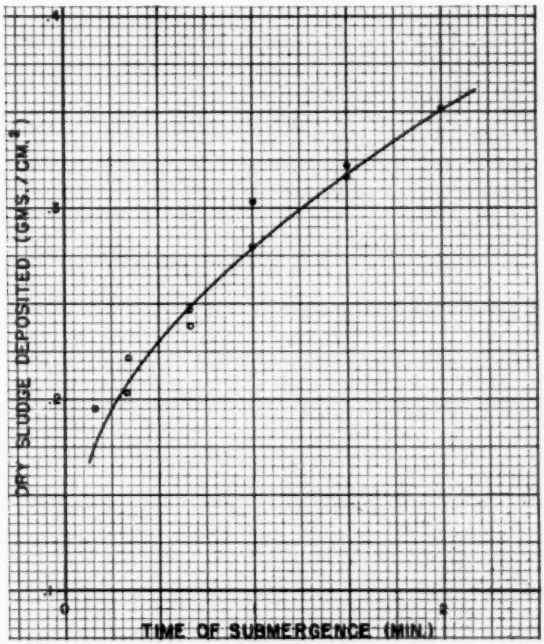

FIGURE 14.—Effect of time of submergence on filter load; Exp. No. 8.

$G = 3.14$, as determined by filter leaf test on this sludge; Col. 2×3.14.

Col. 5.—Sum of Col. 3 and Col. 4.

Col. 6.—Solve Eq. 6, with $c_f = 0.158$ (from filter leaf test); t = time of cycle neglecting time lost in dead space on the filter, chosen as 5 min.; and L_f = value given in Col. 5.

Col. 7.—Read from Figure 13 for corresponding time factors of Col. 6.

Col. 8.—Solve Eq. 7 for $\frac{M_t}{s_t}$.

The final moisture content was found by solving Eq. 8, as shown in the previous problem; M_f = 75.8 per cent.

Thus, the final moisture content, also the minimum which can be obtained with this sludge under the conditions of Experiment 8, is 75.8 per cent. The value of F is given in Col. 7. For example, using the second line and substituting in Eq. 7, $95 = \dfrac{\frac{92.6}{7.4} - \frac{M_t}{s_t}}{\frac{92.6}{7.4} - \frac{75.8}{24.2}}$, or $\frac{M_t}{s_t}$ = 3.6. Also, $M_t + s_t = 1$. Solving, M_t = 78 per cent.

Therefore, the final moisture content of the cake (second line) will be 78 per cent as it comes off the filter. Where per cent filtration is greater than 95, the moisture content is assumed to have reached the minimum or final moisture content.

Col. 9.—Col. $2 \div 5$ min., the time of the cycle.

Col. 10.—Col. 9×123.

The following conclusions may be drawn from the results of these computations, as shown in Figure 15:

1. If the minimum moisture content is to be maintained, little increased yield of dry solids per hour may be gained by shortening the time of cycle, because the minimum practical time of submergence will soon be reached. For example, to maintain 75.8 per cent moisture with a 5-min. cycle, the time of submergence would be 15 sec., an impractical time.

2. Yield may be increased by increasing the time of submergence, but the moisture content of the cake will be increased.

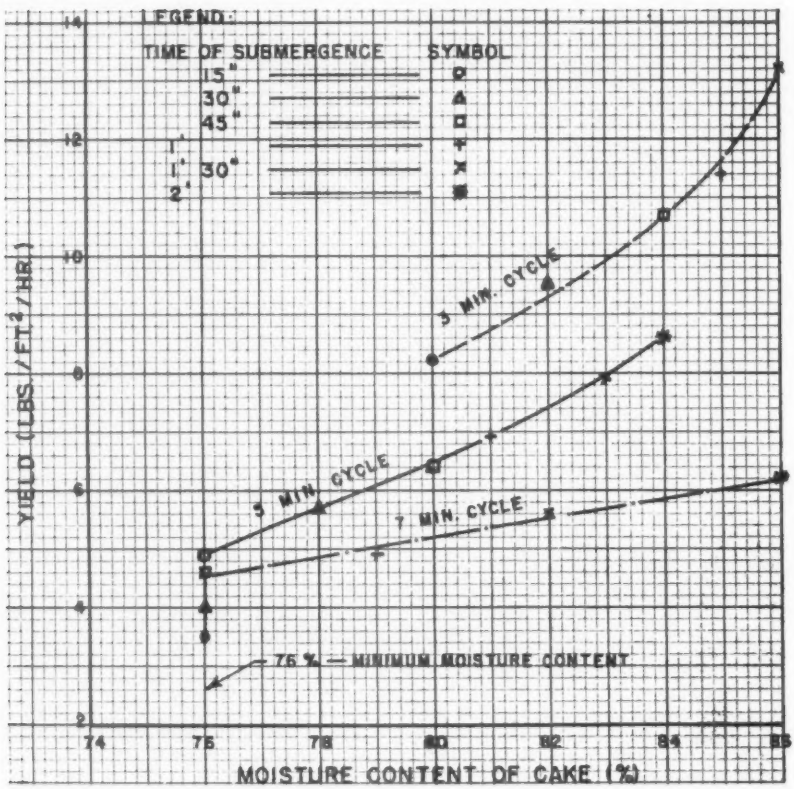

FIGURE 15.—Variation of yield and moisture content with changing time of submergence and time of rotation; sludge of Exp. No. 8.

3. The optimum yield of the driest cake may be obtained with a 6- to 7-min. cycle and a time of submergence of approximately 1 min.

Conclusions

The volume of liquid removed from sewage sludge in the vacuum filtration process follows a definite pattern. If the filtration process is considered as being one involving the removal of liquid from an initial volume of mixed solid and liquid under constant vacuum, the volume of solid will remain constant, while the volume of liquid is gradually reduced until equilibrium is reached. Data from tests which approximated this type of filtration were compared with an equation of the heat conduction type developed by Fourier. The integrated form of this equation is:

$$F = 1 - \sum_{j=0}^{j=\infty} \frac{2}{j^2} e^{-j^2 T} \quad \ldots\ldots (5)$$

in which

F = the amount of liquid removed at any time divided by the amount that will be removed during the entire process;

T = a dimensionless number, which is a function of time, final thickness of cake, and a new coefficient of filtration (c_f). For any filtration

the final thickness of cake and the filtration constant are fixed, thus T = a constant × actual time; and

j = a function of the positive integers.

The relationship between F and T has been determined by solving Eq. 5, thus the time *vs.* volume of filtrate relationship has been established.

This relationship gave good correlation with the data from a modified Buchner funnel test and from a new filter leaf test. Results indicate that the equation fits the data for sludges having a moisture content greater than 5 per cent. More study is needed on the volume of filtrate *vs.* time relationship as the solids concentration in the slurry is varied.

Examination of the relationship between F and T in Eq. 5 indicates that, during the early portion of the filtration cycle, the volume of filtrate varies directly with the square root of time. During the latter portion of the cycle, the volume of filtrate removed is a function of the amount of liquid remaining in the cake.

The results show that sludge, under a given vacuum, is reduced to a constant moisture content, regardless of cake thickness.

Empirical data furnished from the filter leaf test gave the weight of the solids deposited on the filter as a function of time of submergence. From the same data, the coefficient of filtration, c_f, can be determined by curve-fitting methods. These data are all that are necessary to solve operating problems from laboratory experiments. The filtration constant is a function of the type of filter cloth. Until more study shows how this constant is affected by the properties of the filter septum, the same cloth should be used in the test as is to be used on the full-scale filter.

Experiments were performed on digested elutriated sludge dosed with ferric chloride. The filter cloth was a long nap, twill-weave wool. It was demonstrated (a) that if the time of the cycle is variable, then the maximum yield is obtained with the minimum final cake thickness, and (b) that within a certain limit the yield can be improved by increasing the time of submergence without affecting the moisture content. After the limiting point is reached, increasing the time of submergence will increase the yield, but the moisture content will also be increased.

Final acceptance of this hypothesis will depend upon tests with different sludges, under varying conditions, in actual practice.

Acknowledgments

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FIRST QUARTER CONTRACTS TOTAL $27,000,000

Contracts for the construction of 111 sewage treatment plant projects were awarded by municipalities during the first quarter of 1952, according to the U. S. Public Health Service. The projects, distributed in 33 states and involving a total cost of about $27,000,000, will bring new or improved service to more than 2,000,000 people when completed.

Of the 111 projects reported, 72 are new plants being built at a total cost of about $17,000,000. The other 39 consist of additions, enlargements, or replacement of existing plants.

The report is based on data compiled by the Public Health Service from reports of the F. W. Dodge Corporation, state water pollution control agencies, and other construction news sources.

Data are not available on progress in the abatement of industrial wastes, the other major source of water pollution. However, a national summary report published by the Public Health Service in 1950 showed that of the 10,400 industries known to be discharging wastes, nearly 3,500 needed new or additional waste treatment facilities, while more than 5,500 others had undetermined needs.

RADIOACTIVE WASTE DISPOSAL TO PUBLIC SEWERS *

By F. W. Kittrell

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The nuclear reactor at the Oak Ridge National Laboratory was converted from its wartime duties to the production of radioisotopes for peacetime uses in 1946. These materials have unique radioactive characteristics that make them extremely useful tools in research, biology, medicine, agriculture, industry, and other fields. Their value is indicated by the increase in shipments since 1947, the first full year of the program (Figure 1), and by the widespread distribution of users (Figure 2).

The radioactivity which makes radioisotopes such useful tools also renders them potentially hazardous. Radiological damage due to use and disposal of radioisotopes can be avoided through observation of well-established precautionary measures. Members of the sewage works profession are not immediately concerned with potential hazards to personnel using radioactive materials. They are interested, however, in radioisotopes that are discharged to sewers.

A recent excellent article by Van Kleeck (1) on radiation and radioactivity emphasizes the phases that are of interest to the profession. It is a useful reference on background material for a more detailed description of the AEC program for control of potential hazards due to reactor-produced radioisotopes and for illustrative examples of methods for safe disposal of waste radioisotopes into sewers.

* Condensation of paper presented at Fifth Annual Meeting, Kentucky-Tennessee Industrial Wastes and Sewage Works Association; Louisville, Ky.; Sept. 17–19, 1951.

Control of Radiation Safety

AEC Program

Both encouragement of maximum beneficial use of radioisotopes and control of the potential hazards of use are statutory responsibilities of the AEC, set forth in the Atomic Energy Act (2). These responsibilities include radioisotopes produced in foreign reactors and shipped into this country, as well as those produced in this country. The AEC maintains an Isotopes Division to discharge these responsibilities.

The Isotopes Division recognizes the responsibilities of public health agencies for the health of the people in areas under their jurisdictions. It is initiating steps to bring these agencies into closer contact with field investigational phases of the control program.

Allocation

The Isotopes Division is responsible for allocation of reactor-produced radioisotopes used in this country. The allocation procedure has been developed carefully to assure that those who receive radioisotopes are qualified to handle radioactive materials without damage to themselves or to others.

The prospective user submits an application, which includes statements of his experience in handling radioactivity, the instruments available for measuring radioactivity, the kind and amount of radioisotope desired, the use to be made of the material, and the methods to be used for radiation safety. The application is reviewed by a specialist in the proposed use of the radio-

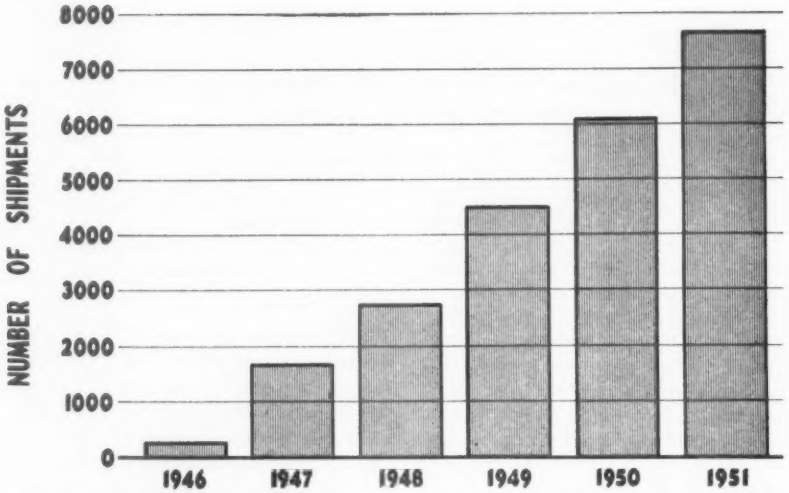

FIGURE 1.—Increase in radioisotope shipments from Oak Ridge National Laboratory, by calendar year.

isotope. If it is indicated that the applicant is fully qualified to handle the material safely, and has adequate facilities for the proposed use, an authorization for purchase of the material is issued him.

The applicant submits a signed statement accepting responsibility for any damage that may result from use of the radioactive material. He also agrees to use the material only for the purpose stated in the application, and

FIGURE 2.—Radioisotope shipments by states, August 2, 1946 to December 31, 1951 (5½ yr.); total number of shipments from Oak Ridge National Laboratory, 22,971.

not to transfer the material to another user without authorization. Thus there is reasonable assurance before shipment that the radioactive material will be handled properly.

Advisory Field Service

In addition to this effective procedure the Isotopes Division maintains an advisory field service. Health safety specialists visit users and offer advice and assistance in the safe use and disposal of radioisotopes. The schedule of visits calls for a visit to each new user within a year after he first receives radioisotopes, and about once every four years thereafter. These visits are comparable to those to officials responsible for municipal water supplies and sewage systems made by members of state health departments. Users' practices, equipment, and records are reviewed with reference to radiological safety. If unsafe practices are found, education and persuasion are used in obtaining improvement. The advisory field service personnel, therefore, serve as consultants rather than policemen.

The authority included in the Atomic Energy Act, however, permits recall of radioisotopes if such a step is deemed necessary. The effectiveness of the combined system, of control through allocation and education through field visits, is attested by the fact that in dealing with more than 1,200 laboratories the recall authority has never been used.

Radioactive Waste Disposal

A number of methods are used for safe disposal of waste radioisotopes. The sewage works profession is only incidentally interested in methods involving burial in the soil or at sea, return to the AEC, and dispersion in the atmosphere. It is, on the other hand, directly interested in dilution and dispersion of these wastes in sewers and subsequently in streams.

An understanding of a few relatively simple concepts involved in the disposal of waste radioactive materials by dilution and dispersion in water will assist greatly in evaluating the degree of hazard involved in any specific situation. These include the unit of measurement of radioactivity, the principle of radioactive decay, and maximum permissible concentrations of radioactive materials.

These radiological concepts are comparable to the everyday principles used regularly in the disposal of industrial wastes in sewers. Other considerations, such as rates of flow of sewage, and dilutions resulting from discharging known amounts of materials into known volumes of sewage are met regularly in the management of sewage systems.

Curie

The curie (c.) is the measure of quantity of radioactive material. It is comparable to pounds of solids or acid, for example, in industrial wastes. It is based on the number of atoms disintegrating per second and is that amount of any radioisotope that provides 3.7×10^{10} disintegrations per second. For convenience, smaller units are used, such as the millicurie (mc.), which is 0.001 c., and the microcurie (μc.), which is 0.000001 c.

Maximum Permissible Concentrations

Maximum permissible concentrations of certain radioisotopes in drinking water and in sewage have been worked out carefully on the basis of available data. These values, adopted by the National Committee on Radiation Protection, include liberal factors of safety. They are comparable to standards governing concentrations of potentially deleterious materials in sewers or streams. An individual limit for each of a number of radioisotopes has been recommended for drinking water for continuous ingestion (normal use for about 30 years) (3), but data are not available at present to permit establishment of individual limits for others. Those for which individual limits have

TABLE I.—Maximum Permissible Concentrations of Radioactivity in Water and Sewage

Radio-isotope	Max. Permissible Conc. (μc./ml.)		Conditions
	Drinking Water	Sewage	
Au^{198}	3×10^{-3}	—	Continuous ingestion[1]
P^{32}	2×10^{-4}	—	Continuous ingestion[1]
Hg^{197}	10^{-7}	—	For unknown or mixed radioisotopes, continuous ingestion[2]
I^{131}	—	5×10^{-4}	Permissible in sewage of institution sewer, based on 24-hr. flow[3]
I^{131}	3×10^{-6}	—	Continuous ingestion[1]
I^{131}	—	10^{-2}	Permissible in sewage of institution sewer if mixed with 1 g. of KI for each μc. of I^{131} [3]
P^{32}	—	10^{-4}	Permissible in sewage of institution sewer, based on 24-hr. flow[3]

[1] Reference (3). [2] Reference (6). [3] Reference (4).

not been recommended are covered by a single limit for mixed or unknown radioactive contamination. This specifies that the concentration of beta and gamma emitters in drinking water shall not exceed 10^{-7} μc. per c.c. of water. A few limits have been established for common isotopes discharged into sewage (4). Selected values are given in Table I.

Half-Life

Radiation decay is a characteristic of radioisotopes that is useful in some waste disposal problems. Radioactivity cannot be reduced or otherwise altered by artificial means, but it does decrease systematically with time. In this respect, it is similar to the familiar B.O.D. The rates of decrease, indicated by their half-lives, differ for different radioisotopes. The half-life of a radioisotope is the length of time required for radioactivity to be reduced to one-half of its initial value. The rate of decrease is constant throughout the life of the radioisotope. If a radioisotope has a half-life of two days, for example, its initial radioactivity will be reduced 50 per cent in two days. In two more days the remaining activity will be reduced another 50 per cent, or to 25 per cent of the initial value. In six days the radioactivity remaining will be 12.5 per cent of the initial. The rate of reduction in first-stage 20° C. B.O.D. could be defined in the same way, with a "half-life" of three days. This characteristic is extremely useful in estimating the residual radioactivity of a material at any time if its initial radioactivity at a given time is known. An approximation may be made on the percentage basis described, or the exact residual may be calculated by use of a formula. Perhaps the simplest method is that using semilogarithmic paper. The initial radioactivity is plotted on the vertical logarithmic axis against zero time on the horizontal axis and 50 per cent of the initial is plotted against the time representing the first half-life. A straight line drawn through these two points gives the residual radioactivity at any time. This is illustrated (Figure 3) for various quantities of radioiodine shipped to three Tennessee cities.

Illustrative Waste Disposal Calculations

Records of actual allocations or shipments of radioisotopes to Kentucky and Tennessee are used as a basis for estimating concentrations of isotopes that may occur in sewers and streams. Generally the largest allocation or shipment to each city is selected as an example. Large allocations usually are received

by the user as a series of several smaller shipments over a protracted period of time. Either water consumption or sewage flow data are available (5) for most of the cities in which radioisotopes have been used. Assumptions have been used rather liberally where certain factors involved in the disposal procedures actually employed are not available.

Kentucky

Of the 1,752 c. of radioisotopes distributed by the AEC since the program started in late 1946, about 3.7 c. have been allocated to the three Kentucky cities of Louisville, Lexington, and Fort

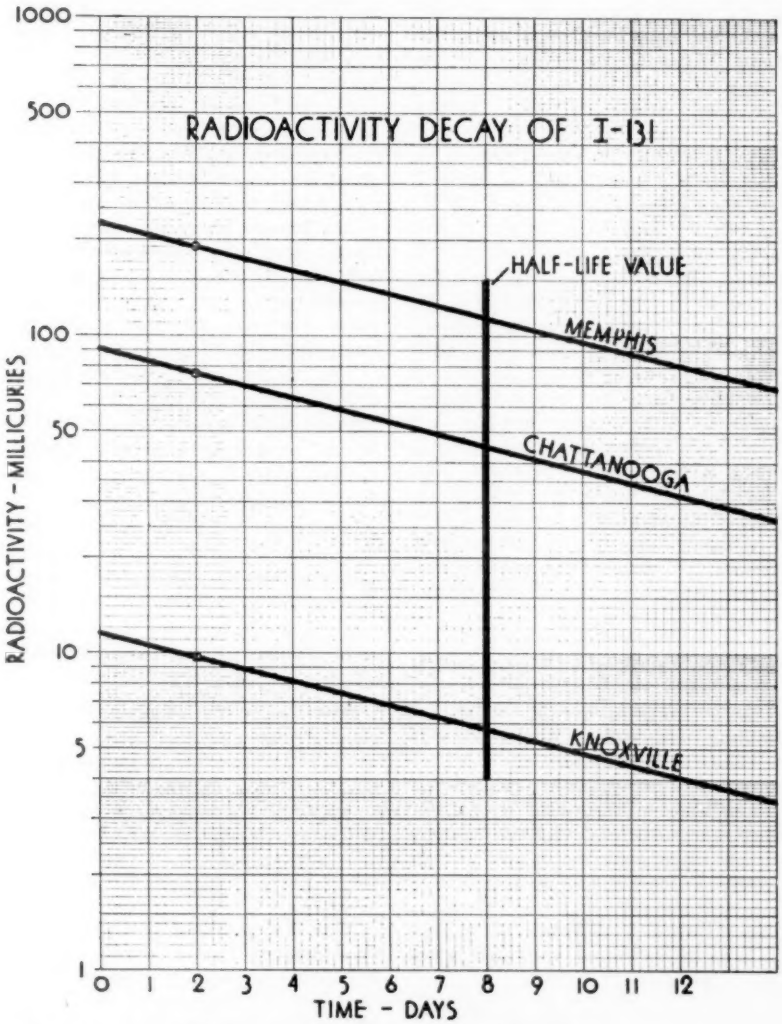

FIGURE 3.—Radioactivity decay of radioiodine.

Knox. Examples given below are based on the largest allocations during the period July 1, 1950 through June 30, 1951.

Louisville: Several types of radioisotopes have been used in animal experimentation at a Louisville institution. Of these, the largest shipment was 200 mc. of Au^{198}, with a half-life of 2.7 days. Because this was used in animal experimentation, only that small portion of the radioactivity unavoidably lost in laboratory manipulation would reach the sewer. An assumption that 10 per cent of the total material was thus wasted should be liberal. The rest would be injected into the experimental animals. If the animals were sacrificed in the experiments, as is likely, their bodies, containing much of the radioactivity, would be disposed of by burial or incineration. For illustrative purposes, it may be assumed that any radiogold wasted to the sewer had decayed through one half-life before discharge. Available data indicate that the total sewage discharge from Louisville is in the neighborhood of 50 m.g.d. Assume that the waste from the institution is diluted by 1 per cent of the total sewage flow of the city before it reaches the river. The radioactivity of the sewage reaching the river may be estimated as follows:

Initial radioactivity = 200 mc. = 2×10^2 mc.

1 mc. = 1,000 μc. = 1×10^3 μc.

Holding time before discharge = 2.7 days = 1 half-life.

Radioactive decay before discharge = 1 half-life = 50 per cent = 5×10^{-1}

Portion of material discharged to sewer = 10 per cent = 1×10^{-1}

Sewage volume available for dilution = 0.5 m.g.d. = 1.9×10^9 c.c.

Concentration of radioactivity in sewage at mouth of municipal sewer

$$= \frac{(2 \times 10^2)(1 \times 10^3) \times (5 \times 10^{-1})(1 \times 10^{-1})}{(1.9 \times 10^9)}$$

$= 5.3 \times 10^{-6}$ μc. per c.c.

Reference to Table I shows that the maximum permissible concentration of Au^{198} in drinking water is 3×10^{-3} μc. per c.c. The estimated concentration of radiogold in the sewage reaching the river is approximately $\frac{1}{600}$ of the limit for drinking water for continuous ingestion, and is completely safe. Other radioisotopes used in Louisville could be disposed of with equal safety.

Lexington: In Lexington several relatively large amounts of P^{32} (in the range of 250 to 725 mc.) were allotted for agricultural experiments. The allotments were for research on fertilizers used on alfalfa, with allocations about three months apart. The radiophosphorus was buried several inches in the ground along with other phosphatic fertilizer. The experimental plot was flat, and precautions were taken to keep out unauthorized persons. The only chance of water contamination here would be the leaching of the radiophosphorus through the soil to some adjacent stream. Preliminary experiments indicate that the rate of leaching of phosphate through soil is about 1 ft. per year. As a matter of interest, assume the most extreme situation possible and calculate how much stream flow would be necessary to dilute the 725-mc. allocation to the maximum permissible concentration for drinking water if an improbable accident permitted the entire amount to reach a stream in a day, before appreciable decay occurred.

Radioactivity = 725 mc.

$= 7.25 \times 10^5$ μc.

Maximum permissible concentration of P^{32} in drinking water = 2×10^{-4} μc. per c.c.

$$\frac{(7.25 \times 10^5)}{(2 \times 10^{-4})(3.8 \times 10^3)} = 9.5 \times 10^5 \text{ gal.}$$

Thus, a stream flowing 950,000 g.p.d., or a little less than 1.5 cu. ft. per sec., would provide safe dilution in the extremely improbable event that the entire amount of radioactive phosphorus reached the stream.

Other allocations to the Lexington institution were used for chemical and physical laboratory research. Storage of radioactive material used in this type of experimental work to permit decay, if necessary, usually is quite simple. The largest allocation was 95 mc. of Hg^{197}, with a half-life of 2.7 days, for use in experimental physics. The total sewage flow for Lexington is 5.0 m.g.d. Assume that the radiomercury, in soluble form, is discharged to the sewer in such a manner that, before it reaches the treatment plant, it mixes with a portion of the total daily sewage flow equal to the average flow for 1 hr. As no individual maximum permissible concentration for Hg^{197} has been adopted, it may be decided to keep the concentration in the sewer below the 10^{-7} μc. per c.c. for mixed or unknown radioactive contaminants in drinking water. First determine the amount of radioactivity that may be discharged to the assumed sewage flow.

Daily sewage flow = 5×10^6 g.p.d.
Average hourly sewage flow

$$= \frac{(5 \times 10^6)(3.8 \times 10^3)}{(2.4 \times 10)}$$
$$= 7.9 \times 10^5 \text{ c.c.}$$

Maximum permissible concentration selected as control = 10^{-7} μc. per c.c.

Maximum permissible radioactivity to sewer = $(7.9 \times 10^5) \times 10^{-7} = 7.9 \times 10$ μc.

The initial 9.5×10^4 μc. of radiomercury could be stored to permit reduction in radioactivity before discharge to the sewer. The portion of the initial that may be discharged to the sewer is

$$\frac{7.9 \times 10}{9.5 \times 10^4} = 8.3 \times 10^{-4},$$

or 0.083 per cent.

Applying 50 per cent reduction for each half-life as previously discussed, it is found that the radioactivity will be reduced to 0.098 per cent in 10 half-lives and to 0.049 per cent in 11 half-lives. The Hg^{197} should be held for 11 half-lives, or about one month, before discharge to the sewer to meet the selected conditions.

Fort Knox: Data on water consumption or sewage flow are not available and no computations are attempted here.

Tennessee

A total of 67.2 c. of radioactivity has been allocated to radioisotope users in Tennessee since the distribution program started. This went to Chattanooga, Johnson City, Knoxville, Memphis, and Nashville, and is exclusive of radioisotopes used in AEC operations at Oak Ridge.

Chattanooga: Although one allocation to a Chattanooga hospital was for 2,000 mc. of I^{131}, records of shipment show that 10 shipments of 100 mc. each were made at monthly intervals. This was for use in diagnosis and treatment of thyroid diseases. Single doses of I^{131} as high as 100 mc. occasionally have been used in cases of advanced cancer of the thyroid, and it is assumed an entire shipment was administered to one patient in this case. A maximum of 90 per cent of the radioiodine administered to a patient may be eliminated in the first day after treatment. (A maximum of 20 per cent of radiophosphorus administered to a patient may be eliminated in the first day.) If all of the patient's body wastes were discharged to the sewer, 90 mc. of radioiodine, adjusted to allow for decay, would be involved. The half-life of I^{131} is 8 days. Assume that the radioiodine was two days old before discharge to the sewer. By use of the semi-logarithmic plotting previously referred to, it is found that the 90 mc. of radioactivity is reduced to 75 mc. in two days (Figure 3). Assume that the hospital has 500 beds and that sewage discharge is 300 g.p.d. per bed, or a total of 150,000 g.p.d.

Thus:

$$\frac{(7.5 \times 10)(1 \times 10^2)}{(1.5 \times 10^5)(3.8 \times 10^3)} = 1.3 \times 10^{-4} \ \mu c. \text{ per c.c.}$$

Table I shows that the permissible concentration under these conditions is 5×10^{-4} $\mu c.$ per c.c., or nearly four times the calculated concentration.

Johnson City: The only radioisotope allocation to Johnson City was a sealed source containing 25 mc. of Sr^{90} for use in treatment of diseased eyes. This is permanent equipment, which is not discarded and therefore presents no waste disposal problem.

Knoxville: There was an allocation of 600 mc. of I^{131} to a Knoxville institution. This was shipped in lots of various sizes, of which one shipment of 115 mc., for animal experimentation, was the largest. It is assumed that 10 per cent, or 11.5 mc., was lost to the sewer in laboratory manipulation. Two days decay would reduce this to 9.6 mc. The total sewage flow from Knoxville, based on water consumption, is about 20 m.g.d. Assume the radioiodine was diluted with 1 per cent of the flow before reaching the river.

$$\frac{9.6(1 \times 10^3)}{(2.0 \times 10^7)(3.8 \times 10^3)(1 \times 10^{-2})} = 1.3 \times 10^{-5} \ \mu c. \text{ per c.c.}$$

Table I shows that the maximum permissible concentration of I^{131} in drinking water is 3×10^{-5} $\mu c.$ per c.c.

Memphis: The largest allocation to Memphis was for 2,500 mc. of I^{131} for diagnosis and treatment of thyroid disorders. Records show that the largest individual shipment was 250 mc. It is extremely doubtful that all of this would be used on patients simultaneously, but can be assumed that it was. With 90 per cent maximum excretion by the patients in the first day after treatment the excreted radioactivity would be that remaining from 225 mc. after allowance for decay. Two days decay, as assumed in other examples, would reduce this to 190 mc. Again assume 150,000 g.p.d. sewage flow for a 500-bed hospital.

$$\frac{(1.9 \times 10^2)(1 \times 10^3)}{(1.5 \times 10^5)(3.8 \times 10^3)} = 3.3 \times 10^{-4} \ \mu c. \text{ per c.c.}$$

This is close to the 5×10^{-4} $\mu c.$ per c.c. permissible under these circumstances. Although this is acceptable, it is so close to the limit that it may be advisable, as a factor of safety, to add potassium iodide to the patient's excreta before discharge to the sewer. This provides what is known as isotopic dilution and allows a higher permissible concentration in the hospital sewer. If 1 g. of potassium iodide is added for each millicurie of radioactivity in the excreta, the permissible concentration of radioiodine at the mouth of the sewer is 10^{-2} $\mu c.$ per c.c. This is 20 times the limit permissible if potassium iodide is not used for isotopic dilution.

Nashville: The largest allotment to Nashville was 10,000 mc. of Au^{198} for tumor studies, but records show that the largest individual shipment was 682 mc. The estimated daily sewage flow for Nashville is about 45 m.g.d., based on water consumption. Assumptions similar to those for the radiogold at the Louisville institution may be used here. Assume one half-life holding time and 10 per cent loss of the radioisotope in laboratory manipulation. Under these conditions 34.1 mc. would be discharged to the sewer. If this were diluted with 1 per cent of the daily sewage flow from Nashville:

$$\frac{(3.41 \times 10)(1 \times 10^3)}{(4.5 \times 10^7)(3.8 \times 10^3)(1 \times 10^{-2})} = 2.0 \times 10^{-5} \ \mu c. \text{ per c.c.}$$

This is far below the permissible concentration of 3×10^{-3} $\mu c.$ per c.c. for Au^{198} in drinking water.

Summary

The Atomic Energy Commission's allocation and visitation program for control of potential radiation hazards

due to off-Commission use of reactor-produced radioisotopes has been described briefly. Sample calculations, based on allocations or shipments of radioisotopes and sewage volumes for Kentucky and Tennessee cities, have been given to illustrate methods of disposal of radioactive wastes in sewers.

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4. "Interim Recommendations for the Disposal of Radioactive Wastes by Off-Commission Users." U. S. Atomic Energy Comm., Isotopes Div., Circ. B-6. Oak Ridge, Tenn. (May, 1946). (Since this paper was presented, the recommendations in Circ. B-6 have been supplanted by the more liberal recommendations of the National Committee on Radiation Protection, Subcommittee on Waste Disposal and Decontamination, in Nat. Bur. of Standards Handbook 49.)
5. "Inventory of Water and Sewage Facilities in the United States, 1945." U. S. Pub. Health Serv., Cincinnati, Ohio (1948).
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ONLY COPPER AND ALUMINUM CONTROLS EASED

Because of the current strike in the steel industry, relaxation of construction controls previously forecast by NPA (see THIS JOURNAL, **24**, 6, 722; June, 1952) has been postponed. By Amendment 1 (effective July 1, 1952) to CMP Regulation 6, however, water and sewage works administrators are able to self-certify increased allotments per project per calendar quarter as follows: copper, up to 750 lb.; aluminum, up to 1,000 lb. Self-certification limits on steel remain at 5 tons, not to include more than 2 tons of structural shapes.

Industrial Wastes

TREATMENT OF COTTON FINISHING WASTES AT THE SAYLES FINISHING PLANTS, INC.*

By George G. Bogren

Partner, Weston & Sampson, Consulting Engineers, Boston, Mass.

The mills of the Sayles Finishing Plants, Inc., are located in Saylesville, R. I., a village in the Providence-Pawtucket area, on the Moshassuck River. The location of the wastes treatment plant in relation to neighboring communities and sewage treatment plants is shown in Figure 1. Adjacent to the wastes treatment plant is the sewage treatment plant of the City of Central Falls.

The Moshassuck River has a drainage area of 7.7 sq. mi. at Saylesville. Storage is provided in several ponds on the drainage area. The river flows in a southerly direction through the cities of Pawtucket and Providence, discharging into the Providence River.

The minimum summer runoff on the Moshassuck at Saylesville is about 2 cu. ft. per sec., or about 1⅓ m.g.d. Because during several summer months the water consumption of the company mills exceeds the runoff, there is no dilution of the wastes discharged.

Saylesville is about 5 hr. river flow, at low stage, from tide water in the Providence River. This is an important factor in setting the standard of quality of wastes discharged at Saylesville.

Volume and Character of Wastes

The present volume of wastes is estimated to total 5.1 m.g.d., of which about 2.5 m.g.d. are discharged to the existing treatment plant. Alterations to the wastes collecting system at the mills will increase the volume discharged to the proposed treatment plant to about 3.3 m.g.d., the balance being discharged to the river without treatment. Table I shows the average composition of the mixed wastes, the wastes separated and discharged to the present treatment plant, and the estimated character of the wastes separated and discharged to the proposed sewage treatment plant. It will be seen that the wastes are highly alkaline with a caustic alkalinity, on the average, and a B.O.D. slightly lighter than average domestic sewage. After more complete separation, the wastes discharged to the proposed treatment plant will be less alkaline, free of caustic alkalinity, and slightly lower in B.O.D. and suspended solids, due to the inclusion of certain light but highly acid wastes.

TABLE I.—Average Character of Finishing Plant Wastes

Item	Mixed Wastes	Present Treatment Plant Influent	Est. Future Treatment Plant Influent
Turbidity (p.p.m.)	125	250	225
Color	(Dyes)	(Dyes)	(Dyes)
Oxygen cons. (p.p.m.)	300	350	325
Alk., total (p.p.m.)	500	600	325
Alk., hydroxide (p.p.m.)	100	175	0
pH	9.0	11.0	7.5
Susp. solids (p.p.m.)	100	200	200
Settl. solids (%)	0.25	0.50	0.45
B.O.D. (p.p.m.)	175	225	200

The wastes include the discharge of mill toilets and the sewage from a portion of the village of Saylesville,

* Presented at 1951 Annual Meeting, New England Sewage Works Assn.; Providence, R. I.; November 6–7, 1951.

amounting to about 10 per cent of the total volume of heavy wastes separated for treatment. The character of the wastes varies widely in color and chemical composition from hour to hour, as well as in rate of discharge.

Because of the presence of biological inhibitors, such as caustic, acid,

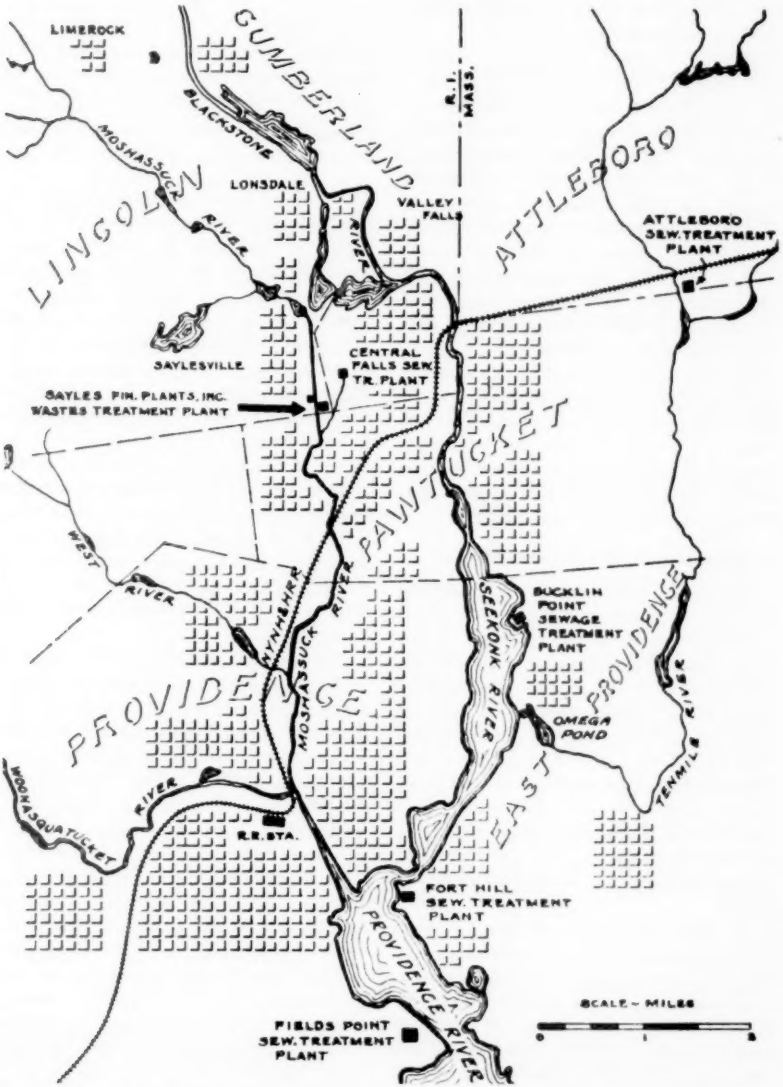

FIGURE 1.—Location plan of Sayles Finishing Plants, Inc., wastes treatment plant to show relation to other treatment plants on watershed.

hypochlorite, and peroxide, the wastes discharged to the treatment plant are sterile much of the time, and their B.O.D. accordingly is slow to develop; the 24-hr. B.O.D. is about one-eighth that of domestic sewage, and the 12-hr. B.O.D. about one-twenty-third.

Separation of Wastes

The mills are served by two systems of sewers or drains, one discharging so-called "heavy" wastes to the treatment plant and the other discharging "light" wastes directly to the river. Recently an extensive sampling of the discharge of the individual processes was a basis for the modernization of these systems. The future volume and character of wastes to be treated has been estimated on this basis.

In some plants the cost of separate sewer systems is not warranted, but in this case separation is desirable, not only to reduce the size of treatment units, but also to permit closer control of the alkalinity of the wastes discharged to the treatment plant.

History of Treatment Works

In 1920 a collecting sump, pumping station, primary settling basins, and sludge beds were constructed. The three manually-cleaned settling basins, rectangular in shape with sloping bottoms, have a total volume of 782,000 gal. They will provide a detention period of 5.6 hr. under the proposed average future flow, and 2.4 hr. at peak future flow; when constructed they provided a detention period of about 4.0 hr. at peak flow. There are six sludge beds having a total area of 0.54 acres.

In 1923 plans were drawn for complete treatment of the settled wastes. These provided for four dosing tanks with automatic siphons, four trickling filters having a total area of 1.0 acre, and four intermittent fine-cinder filters having a total area of 4.0 acres. The dosing tanks, 0.25 acre of trickling filter, and 1.0 acre of intermittent cinder filter were constructed in 1924, and have been operated continuously since. Table II shows the average results of operation of the present plant, expressed as per cent removal of the principal polluting constituents by the three units of the treatment plant. The percentages are cumulative, as they refer to the raw wastes analysis in each case. It will be seen that because of the non-settling nature of the wastes, the settling basins remove far less suspended solids and B.O.D. than is common practice for domestic sewage. Nevertheless, the total volume of dried sludge removed from the sludge beds per year averages about 1,000 tons.

TABLE II.—Operation Results of Present Plant

Item	% Removal by			
	Primary Basins	Trickling Filter	Cinder Filter	Entire Plant
Turbidity	15.7	18.6	64.4	98.7
Oxygen consumed	9.1	25.3	53.0	87.4
Suspended solids	17.4	11.7	70.5	99.6
B.O.D.	15.5	36.4	40.8	92.7
Settleable solids	96.5	0.0	3.5	100.0

The present trickling filter consists of 10 ft. of coarse stone varying in size from 1 in. to 2 in. Its removals, like those of the settling basins, are less than in the case of domestic sewage. It is surprisingly little affected by extreme variations in character of the settling basin effluent of short duration, and produces an effluent with a pale straw color regardless of the color of its influent. Frequent unloading of biological accretions has indicated the need of a settling basin for its effluent.

The intermittent cinder filter was constructed of this material because of the availability at the site of a large quantity of cinders. As will be seen from Table II, its performance is remarkable. It is dosed intermittently

by a pump, and handles trickling filter effluent at a rate averaging 0.3 m.g.a.d.

Recent Developments

Shortly after World War II the Blackstone Valley Sewer District was created, and plans were subsequently drawn for the construction of intercepting sewers and treatment works. The Sayles finishing plants are within the geographical limits of the District. At the recommendation of the District's consulting engineers, however, the wastes from the company's mills are not to be taken into the District's system, but are to be given complete treatment by the company. This was because of the fact that the company already had a considerable investment in wastes treatment facilities, and because of the adverse effect of these wastes upon the size and type of treatment plant required for the District.

The State Department of Public Health, in its report on water pollution dated December 23, 1946, classified the Moshassuck River below Saylesville as Class D, the lowest classification above that of an intolerable nuisance. The requirements of this classification are the presence of dissolved oxygen in the water at all times, and freedom from odors, slick, or visible floating solids. The aim of pollution abatement on this river is primarily to eliminate odors from it.

As soon as the company was aware that it was not to be included in the Blackstone Valley Sewer District, it proceeded with plans for complete treatment of its wastes. A pilot plant had been constructed to determine the feasibility of high-rate trickling filters, as the cost of constructing and maintaining intermittent cinder or sand filters was relatively prohibitive. The

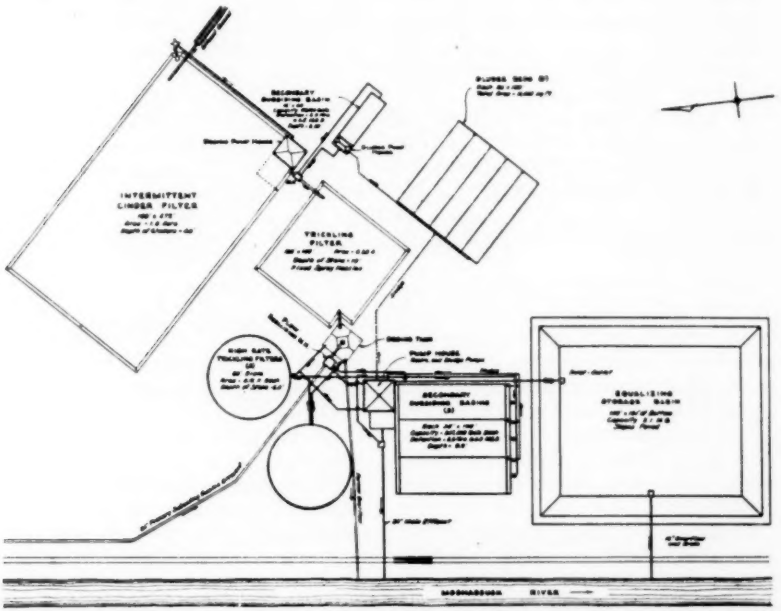

FIGURE 2.—Layout of finishing wastes treatment plant, including old and new units.

pilot plant studies indicated that the settling basin effluent could be successfully handled on high-rate trickling filters at B.O.D. loads comparable with similar filters handling domestic sewage and with an estimated over-all B.O.D. reduction of at least 60 to 70 per cent (1). Studies were thereupon made for the separation of wastes at the source to reduce the alkalinity of the wastes to be treated so that the efficiency of the high-rate trickling filters could be increased.

Proposed Additional Treatment Works

Construction of additional works to provide trickling filter treatment and secondary clarification of the entire volume of settling basins effluent has begun. It is proposed to utilize the existing trickling filter and intermittent cinder filter to their capacity, estimated at 0.3 m.g.d. An estimated 3.0 m.g.d. will be handled by new trickling filters. The proposed plant addition consists essentially of the following parts (see Figure 2).

1. An equalizing basin to store discharges of wastes from the settling basins in excess of the capacity of the old and new trickling filters. It has a capacity of 2.1 m.g., asphalt-paved earth side slopes, and an unpaved earth bottom.

2. Two high-rate trickling filters each 83 ft. in diameter, having a total area of 0.24 acre. They will have concrete walls, surrounding a 6-ft. depth of 2- to 3-in. stone. The wastes will be applied through rotary distributors. The filters are designed for a B.O.D. loading of about 2.0 lb. per cu. yd. of stone, and a 1:2 recirculation ratio is provided.

3. Three secondary settling basins, each 35 ft. by 105 ft., and each having a capacity of 247,000 gal., each providing a detention period of 2.0 hr. at a 3.0-m.g.d. rate of flow. They are cleaned by mechanical sludge scrapers.

TABLE III.—Anticipated Results of Operation of Proposed Plant

Item	% Removal by			
	Primary Basins	High-Rate Tr. Filter	Sec. Settl. Basin	Entire Plant
Oxygen consumed	25	35	5	65
Susp. solids	30	0	45	75
B.O.D.	30	40	5	75
Settl. solids	98	0	—	98

4. A secondary subsiding basin for the existing trickling filter, 16 ft. by 60 ft. in plan, having a capacity of 45,000 gal. and providing a detention period of 2.0 hr. at 0.5 m.g.d. It is provided with a mechanical sludge scraper.

5. Five sludge beds having a total area of 10,000 sq. ft., to serve the secondary settling basins.

6. A flow equalizing manhole and a pump house to maintain a constant discharge to the trickling filters, and to provide recirculation. In the flow equalizing manhole is a long weir, over which the flows in excess of the capacity of the trickling filters discharge to the equalizing storage basin. In the pump house are two 3.0-m.g.d. pumps with variable discharge. When the flow of sewage from the settling basins falls below the rated capacity of the trickling filters, these pumps will draw stored wastes from the equalizing basin to maintain the rate of application to the trickling filters. The speed of the pumps is controlled by a sensing device actuated by the liquid level in the equalizing manhole. There are three 3-m.g.d. recirculating pumps.

Two vacant compartments of the existing dosing tank are to be utilized as head chambers to feed the two new high-rate trickling filters, and a third is to continue to serve the existing trickling filter. The fourth compartment is available for an additional unit, if required.

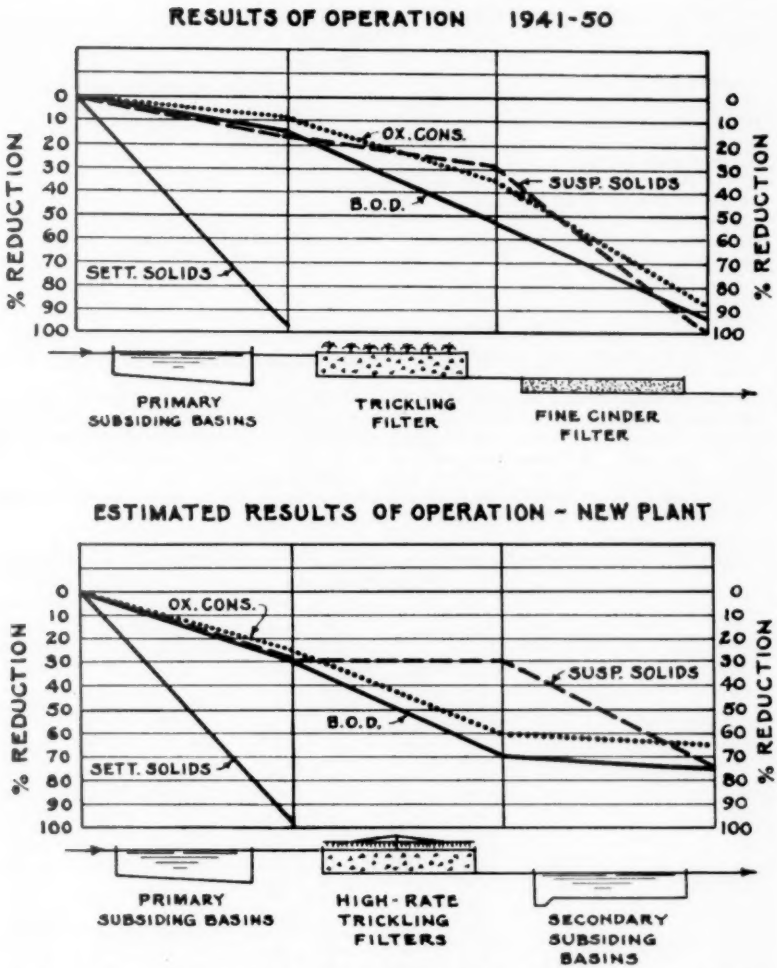

FIGURE 3.—Comparison of estimated operation results for new plant with actual 10-year results with old plant.

Piston-type sludge pumps discharge the sludge from the secondary settling basins to the sludge beds. A Venturi meter ahead of the flow equalizing manhole will indicate and record the sewage flow.

Table III gives the anticipated results of operation of the proposed plant. Figure 3 also gives estimated future results of operation compared with the results for the past 10 years with the present plant. It will be seen that the expected future removals of suspended solids and B.O.D. are considerably greater than those of the present trickling filter, but considerably below those of the present cinder filter. The estimated character of the

proposed treatment plant effluent is as follows:

Turbidity (p.p.m.)	—
Color	Light
Oxygen cons. (p.p.m.)	115
Alk., total (p.p.m.)	300
Alk., hydroxide (p.p.m.)	0
pH	7.0
Susp. solids (p.p.m.)	50
Settl. solids (%)	0.01
B.O.D. (p.p.m.)	50
Diss. oxygen (p.p.m.)	2.0

Because of the chemical constituents in these wastes, already mentioned, the effluent should be more stable, or less subject to decomposition, than the effluent from a similar plant handling domestic sewage.

Reference

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CALIFORNIA SEWAGE TREATMENT PLANT COSTS

In a paper on "Sewage Disposal in the San Joaquin Valley," E. S. Cary, associate sanitary engineer, and H. B. Foster, Jr., supervising sanitary engineer, California State Board of Health, have compiled some interesting data (Table I) relating to sewage treatment plant construction costs in that area. The paper was presented at the 1952 Annual Meeting of the California Sewage and Industrial Wastes Association, held at Santa Cruz, Calif., April 23–24, 1952.

The table gives the accepted bid costs of the several plants, without any attempt to pro-rate them according to construction indexes.

TABLE I.—Bid Costs for Construction of Sewage Treatment Plants in San Joaquin Valley, Calif.

City	Year Const.	Population		Design Flow (m.g.d.)	Type[1]	Bid Cost on Plant[2] ($)	Cost per Cap. ($)	Cost per M.G. ($)
		Present	Design					
Fresno	1948	90,626	225,000	20.0	Prim.	677,595	3.01	33,900
Bakersfield	1951	54,540	98,000	16.0	Prim.	938,102	9.57	58,600
Merced	1951	15,001	35,000	3.5	Prim.	160,262	4.58	45,800
Hanford	1948	9,829	20,000	2.41	Int.	203,514	10.17	84,400
Mt. Vernon	1948	18,000	18,000	2.0	Sec.	274,800	15.27	137,400
Oildale[3]	1947	20,000	16,000	2.0	Sec.	215,000	13.44	107,500
Tulare	1949	12,367	15,000	1.5	Int.	150,240	10.01	100,100
Taft	1951	3,678	12,000	0.72	Int.	129,791	10.41	180,800
Selma	1949	5,934	12,000	1.8	Int.	111,293	9.28	62,000
Sanger	1949	6,357	9,000	1.15	Prim.	166,933	18.55	145,200
Kingsburg	1948	2,298	6,000	0.87	Sec.	91,870	15.31	106,000
Avenal	1951	4,000	6,000	0.60	Prim.	59,307	9.89	98,900
McFarland	1949	1,000	4,000	0.4	Int.	119,000	29.75	297,000
Clovis	1948	2,757	3,500	0.28	Prim.	45,500	13.00	163,000
Fowler	1951	1,859	3,500	0.35	Prim.	44,980	12.84	128,400
Woodlake	1951	2,512	3,500	0.21	Prim.	27,500	7.86	131,000
Kerman	1951	1,800	3,000	0.30	Prim.	34,980	11.65	116,500
Mendota	1948	1,537	3,000	0.3	Prim.	46,912	15.61	156,000
Armona	1949	1,200	2,000	0.148	Prim.	37,407	18.70	253,000
Pixley	1950		2,000	0.2	Prim.	54,663	27.33	273,000
Strathmore	1948		2,000	0.2	Prim.	42,550	21.30	213,000
Tranquility	1950		1,500	0.15	Prim.	21,745	14.50	145,000
Kern County Gen. Hosp.	1949		500	0.0625	Prim.	17,000	34.00	272,000
Camp Owens	1950	150	150	0.0075	Prim.	13,749	91.66	1,833,000

[1] Primary plants include Imhoff tanks or clarifiers with separate digesters; intermediate treatment consists of primary clarifiers, plus either standard or high-rate filters; secondary treatment consists of primary clarifiers, high-rate filters, recirculation and secondary clarifiers with separate sludge digestion, or high-rate activated sludge.

[2] Land and engineering costs not included.

[3] North of River District.

WASTE LOADINGS FROM POTATO CHIP PLANTS

By Ralph Porges

Senior Sanitary Engineer, Missouri Drainage Basin, U. S. Public Health Service, Kansas City, Mo.

Numerous small industries frequently encountered during stream pollution surveys may produce significant amounts of liquid organic wastes. It becomes necessary, therefore, to estimate the pollution discharged, either by investigation of the individual establishment or by applying unit figures secured from studies of similar plants. The latter method may save considerable time and is the only alternate where a field staff and laboratory facilities are not available for a complete industrial waste appraisal.

Potato chip manufacture is one of these small industries. Meager information regarding wastes from this process made it desirable to conduct somewhat limited studies for establishment of production unit pollution factors to assist in evaluation of such plants. The analytical data on which this report is based are limited to one potato chip plant and therefore may not give extremely accurate indications of waste loadings from these plants. Nevertheless, the data will be useful in many cases where an estimate of the approximate waste loading is desirable.

Method

Studies were made over the operating cycle of a potato chip plant. Production was obtained from plant management and waste characteristics determined so that unit factors could be developed.

Laboratory analyses included the following: B.O.D. (standard 5-day 20° C.); suspended solids (total, fixed, and volatile); and settleable solids (1-hr.). All analyses were conducted in accordance with "Standard Methods" (1).

Description of Process

Potatoes were received in 100-lb. sacks and stored prior to use. They were then peeled, washed, sliced, soaked, and finally cooked in oil.

Potatoes were supplied on a batch basis to a peeler consisting of revolving carborundum wheels, which removed the outer portion of the potato. After peeling, the potatoes were washed, inspected, and eyes and defects removed. They were then conveyed to the slicer. The potatoes were kept wet during handling to prevent discoloration. After slicing, the slivers of potatoes were washed and soaked in a vat to remove starch and surface material which might cause sticking and matting of the chips. The water was removed by a centrifugal dryer and the slices cooked in oil. After cooling, excess oil was removed by another centrifugal dryer and returned to the cooking vat and the chips packaged.

Operational data from several plants are shown in Table I. The chips pro-

TABLE I.—Potato Chip Plant Operation

Plant	Employees[1] (no.)	Potato Chips[1] (lb.)	Oil Used[1] (lb.)
A	2.5	250	150
B	4.0	250	125
C	5.8	—	—
D	2.0	300	—
E	3.8	250	190
F	5.0	220	—
Av.	3.8	254	153

[1] Per 1,000 lb. of potatoes processed.

duced from 1,000 lb. of potatoes averaged about 250 lb.

Sources of Waste

Peeler

Water was fed continuously during the peeling operation, the volume varying with the amount of potato removed. The latter is highly variable, depending upon smoothness, depth of eyes, and regularity of shape. Rubbed-off material was discharged with waste water.

Washer

After slicing, the slivers were washed and soaked in a vat to remove starch and other surface material. The vat, equipped with an agitator, had a continuous water feed, the overflow discharging to a floor drain.

Miscellaneous

Oil spills on the floor around the cooking vat were washed to the floor drain. Wash-up of equipment and floors also contributed some waste. Storage wastes, consisting of decayed potatoes, eyes and sprouts, approximated 2.5 per cent and were disposed of to a dump.

Quantity of Waste

The volumes of liquid wastes are shown in Table II. During the period of this study, the peeler utilized 234 gal. and the washing and soakage tank 320 gal. of water per 1,000 lb. of potatoes. This totals 554 gal. per 1,000 lb. of potatoes, although maximum use was reported as 1,000 gal.—a variation caused by the type and shape of potato processed. Another plant reported an average 810 gal. per 1,000 lb. of potatoes processed.

Character of Wastes

Analyses of wastes from peeler and soakage tanks are also shown in Table II. It is of interest to note the small percentage of fixed suspended solids, indicating the high organic content of starch waste. Strength of the waste is several times that of domestic sewage. During this study, which did not represent average conditions, the sewered population equivalent per 1,000 lb. of potatoes processed was 39 based on B.O.D. and 65 based on suspended solids.

Discussion

Characteristics of the type of potato being processed will affect the quantity and quality of waste produced. The rougher the potato, the more will be removed to produce a product suitable for slicing. From about 7 to 30 per cent of the potato may be removed during peeling; average conditions will approximate 20 per cent.

The solids content of potatoes may vary from about 15 to 30 per cent; the Idaho type has the highest content and Rocky Mountain varieties the lowest. Eastern varieties are intermediate (2). Other factors, such as seasonal variation, storage, and method of growing,

TABLE II.—Potato Chip Processing Wastes per Production Unit of 1,000 Pounds Potatoes

Process	Waste Flow (gal.)	B.O.D. (p.p.m.)	Sett. Sol.[1] (ml./l.)	Suspended Solids			Pop. Equiv.	
				Total (p.p.m.)	Vol. (p.p.m.)	Fixed (%)	B.O.D.	S. S.
Peeling and washing	234	580	18	1,120	1,004	10.3	6.8	10.9
Washing and soakage tank	320	2,020	—	4,068	4,024	1.1	32.3	54.3
Total	554	—	—	—	—	—	39.1	65.2

[1] 1-hr. settling.

TABLE III.—Waste Evaluation

Per Ton of Potatoes	Peeler Waste		Soakage Tank Wastes	Spray Washing Wastes[3]
	For Chips[1]	For Dehydration[2,3]		
Flow (gal.)	468	600	640	2,500
B.O.D. (p.p.m.)	580	4,000	2,020	1,000
B.O.D. (lb./ton)	2.264	20	10.8	20
B.O.D. (lb./ton/% loss)	0.32	1.0		
B.O.D. (lb./lb. dry potato solids lost)	0.11	0.20		
Suspended solids (p.p.m.)	1,120	18,000	4,068	1,500
Suspended solids (lb./ton)	4.37	90	21.7	30
Suspended solids (lb./ton/% loss)	0.63	4.5		
Suspended solids (lb./lb. dry potato solids lost)	0.22	0.90		

[1] 7 per cent loss; 15 per cent potato solids.
[2] 20 per cent loss; 25 per cent potato solids.
[3] From reference (2).

also influence solids composition; 25 per cent solids may be taken to represent the average. At the time of this study, California White Shafter potatoes from the new crop were used, resulting in only 7 per cent loss through the peeler. The solids content was 15 per cent. The plant manager stated that with older potatoes the amount of peelings removed might more than double to obtain a smooth clean product suitable for slicing.

A more useful value for evaluating the amount of pollution normally discharged by a potato chip plant would be that based on an average type of potato—25 per cent solids and 20 per cent loss through peeling operations. To arrive at such a value present data were calculated on a solids basis (Table III). The pounds B.O.D. in the peeling waste per pound of dry potato solids was 0.11 and, similarly, the pounds suspended solids discharged per pound of dry solids wasted was 0.22. Applying these values to average conditions as given previously, the population equivalent of peeler wastes from 1,000 lb. of potatoes processed would be 33 based on B.O.D. and 66 on suspended solids. Wash and soakage wastes (Table II) gave population equivalents, per 1,000 lb. of potatoes handled, of 32 for B.O.D. and 54 for suspended solids. This results in a total waste equivalent for each 1,000 lb. of potatoes of 65 based on B.O.D. and 120 based on suspended solids.

These data were compared with potato dehydration wastes presented by Gray and Ludwig (2). Because the suspended solids (Table III) and settleable solids (185 ml. per l.—1 hr.) of peeler waste from the preparation of potatoes for dehydration ran much higher than in potato chip preparation wastes, it appears that the potato received rougher treatment in the former process, thereby causing the potato solids to be larger and measured as settleable and suspended solids. Referring to Table III, it will be noted that pounds of B.O.D. per pound of potato solids lost in potato chip peeler operation was 0.11 as compared to 0.20 lb. with a similar process preparing potatoes for dehydration.

Potato chip soakage tank waste indicated 10.8 lb. B.O.D. per ton, as compared to 20 lb. from spray washing operation prior to dehydration. Likewise, the pounds of suspended solids per ton of potatoes processed were 21.7 for this study and 30 for the dehydration wastes. The correlation of wastes from the potato chip process and potato dehydration preparation appears reasonable, considering the wide varia-

tion in types of potatoes and methods of processing.

Summary

A study was made over the operating cycle of a potato chip plant to determine unit production pollution factors. Based on potatoes having 25 per cent solids content and 20 per cent weight loss through peeling operations —an average condition—the following appear to be reasonable estimates for appraising potato chip processing wastes:

Type of Process	Pop. Equiv. per 1,000 lb. Potatoes B.O.D.	Susp. Sol.
Potato peeling	33	66
Soakage and wash	32	54
Total	65	120

Acknowledgment

The study reported herein was conducted as part of a water pollution investigation of the Lower Platte River Basin developed under a cooperative agreement among the Nebraska State Department of Health, the U. S. Bureau of Reclamation, and the U. S. Public Health Service.

Special acknowledgment is made to the Division of Sanitation, Nebraska State Department of Health, and the field staff participating in these studies, particularly chemists W. W. Walker and C. N. Shadix of the U. S. Public Health Service. In all instances, cooperation by industry was excellent.

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NEW ENGLAND INDUSTRIAL WASTES INVENTORY

"Industrial Wastes in the New England Interstate Water Pollution Control Compact Area," a comprehensive summary of two detailed industrial wastes surveys conducted under the sponsorship of the Commission, has recently been issued. The surveys were made to determine the kind, location, and concentration of industrial plants, the various types and quantities of industrial waste effluents, and their disposal. The report is designed to present an over-all picture of the industrial waste problem in the area and the relative importance of each industry in the pollution control program.

The survey data were obtained from state water pollution control agencies, city and town engineers in some of the larger manufacturing communities, and officials of many industrial plants. Included in the surveys were 2,775 plants, of which 1,939 had processes resulting in the discharge of liquid wastes.

The surveys were conducted and the report prepared by Professor Henry Campbell, head, Dept. of Civil Engineering, University of Rhode Island, assisted by the staff of the University's Engineering Experiment Station.

RAG, ROPE, AND JUTE WASTES FROM SPECIALTY PAPER MILLS *

V. Treatment by Aeration

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More energy is liberated from aerobic decomposition of organic matter than from anaerobic decomposition, hence, breakdown of the organic compounds into simple gases and water is more complete in aerobic processes. It follows that the resulting B.O.D.'s of aerobic treated wastes should be considerably lower than anaerobic effluents after equal periods of decomposition, provided a waste is amenable to aerobic treatment. A convenient method for determining whether a waste is amenable to aerobic biological stabilization is to measure directly the oxygen utilized by the waste solution.

The objects of the studies reported in this paper were to determine the effect of a number of environmental factors on the rate of oxidation of cooking and wash water waste mixtures, and to determine the degree of oxidation obtainable under optimum conditions with flocculent-growth (sludge) and dispersed-growth aeration methods.

Nutrient Addition

Because the cooking liquors and wash waters are deficient in nitrogen and phosphorus, sufficient diammonium acid phosphate was added to maintain a B.O.D.:N:P ratio of 20:1:1. The B.O.D. of mixtures of cooking liquors and first-hour wash water were about 2,000 p.p.m.

pH

Direct oxidation studies showed that at a pH of 9.0 or below, rapid oxidation occurred, but practically none occurred above pH 10.0 (Figure 1). Continued aeration of the rope waste, adjusted to pH values of 9.0, 8.0, and 7.0, resulted in a comparatively rapid change of pH and an equilibrium of about pH 8.5 after 48-hr. aeration (Figure 2). This seems to indicate that a pH value of 8.5 constitutes optimum conditions for aerobic treatment of the mixture of rope and wash-water waste.

Waste Concentration

Experiments with various concentrations of wastes showed that a waste concentration of at least 2,000 p.p.m. B.O.D. (85 per cent of total strength) could be readily oxidized. This is illustrated in Figure 3, where an 85 per cent waste concentration is compared with a 50 per cent concentration.

Seed Adaptation

To obtain maximum oxidation of the organic matter in the waste, the organisms responsible for the oxidation must be adapted to the food and environment. The effect of adapted seed is illustrated in Figure 4. The adapted seed produced a higher rate of oxidation than did the unadapted seed. The quantity of seed material also affects the rate of rope waste oxidation. To determine the minimum quantity of seed material required, various percentages of seed were added. The results (Figure 4) show that, in general, increased quantities of seed resulted in increased rates of rope waste oxidation, but that the increase with seed addition

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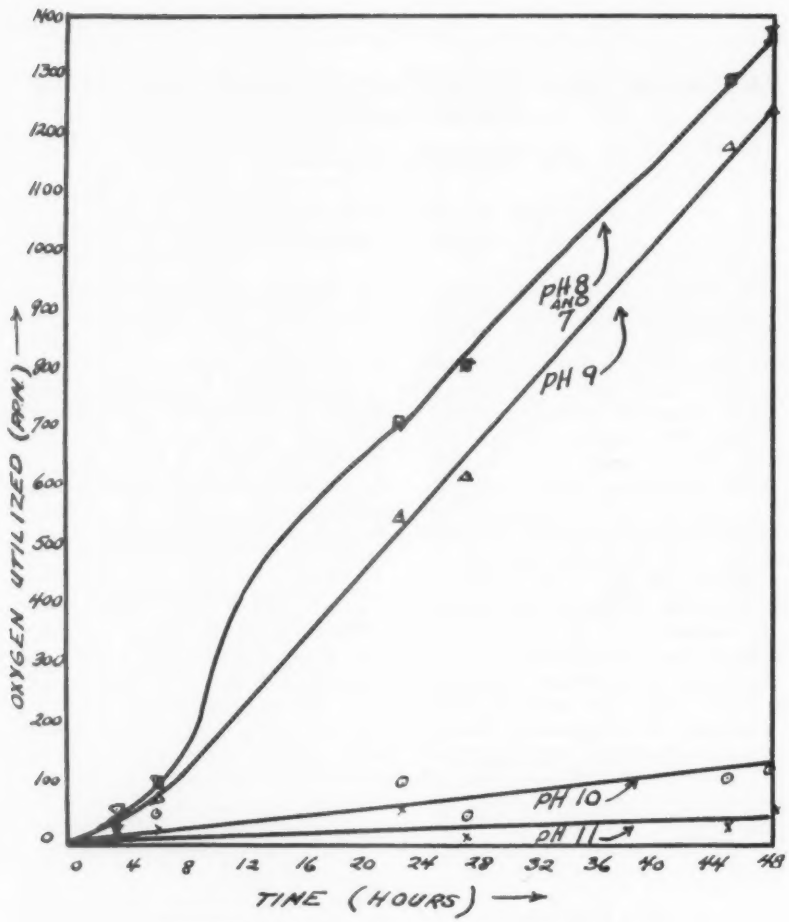

FIGURE 1.—Effect of pH on oxidation of rope waste.

of more than 25 per cent by volume is comparatively small.

Dispersed-Growth Aeration

Heukelekian (2) states that the treatment of strong wastes by the activated sludge process is difficult. The sludge disperses or bulks. The treatment process by nonflocculent growths (or dispersed-growth aeration) consists of inoculation of the waste with a soil or other suitable bacterial suspension and aeration. At the end of the aeration period, settleable solids, if produced, are settled, and a portion of the settled liquor is retained for seeding purposes. He further states that the B.O.D. reduction by this process depends upon the B.O.D. of the raw waste and the time of aeration. He found 80 per cent B.O.D. reduction in 24 hr.

with wastes containing an initial B.O.D. of up to 3,000 p.p.m. using air at the rate of 2 cu. ft. per gallon of waste per hour.

Numerous experiments conducted with dispersed growth aeration of rope waste at 20° C. resulted in B.O.D. reduction of 60 to 70 per cent in 24 hr. The average results of 7 experimental runs with an air supply of 2.75 cu. ft. per gallon per hour at a temperature of 20° C. illustrates this point (Figure 5). No matter how well the seed was adapted, the B.O.D. reduction did not exceed 72 per cent.

The method of seed adaptation consisted of gradually decreasing the quantity of fresh sewage added to the aerator each day and increasing proportionately the quantity of returned supernatant seed. The adaptation period was usually continued for at least one week, with daily settling and return of supernatant and fresh sewage additions to new rope waste. A very active dispersed-growth seed was developed in this manner.

Air Rate

The maximum of B.O.D. reduction in the presence of dispersed-growth seed after 24-hr. aeration was obtained with an air rate of 2.75 cu. ft. per gallon of waste per hour at 20° C. When the air rate was lowered to 2 cu. ft. the same reduction in B.O.D. was obtained, but when the air rate was lowered to 1.1 cu. ft. the B.O.D. reduction was decreased to 67.5 per cent. Further reduction of air decreased the percentage B.O.D. reduction, but not in proportion to the reduction in air (Figure 6). From the results it appears that to obtain maximum B.O.D. reduction the air supply must be in the neighborhood of 2 cu. ft. per gallon per hour, but that the air consumption can be cut to as low as one-fourth of the optimum with a resulting sacrifice of about 10 per cent in B.O.D. removal. The results

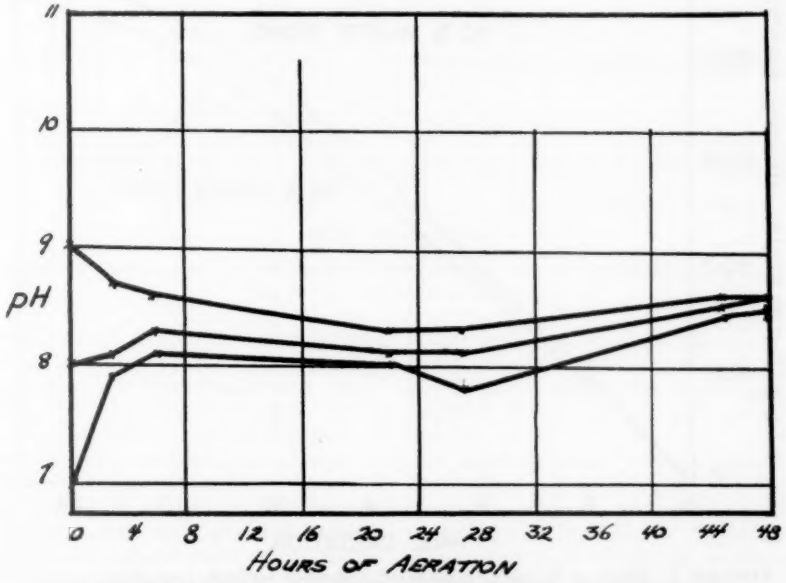

FIGURE 2.—Change in pH values during oxidation of rope waste.

further indicate that it is comparatively easy to remove some 60 per cent of the B.O.D., and, if warranted, considerable economy in air consumption can be made.

Temperature

Several separate experiments conducted at 35° C., in the presence of dispersed-growth seed and a 24-hr. aeration period with an air rate of 2 cu. ft. per gallon per hour, showed that this treatment process consistently reduced the B.O.D. at least 85 per cent. This increase from 60 to 70 per cent reduction at 20° C. to 85 per cent reduction at 35° C. is sufficient to make this method of treatment practical. When compared to anaerobic digestion of rope waste, the detention period required is one-sixth as long and the B.O.D. reduction about 15 per cent higher, with an effluent B.O.D. of from 300 to 350 p.p.m.

Sludge Seeding

The return of the settled sludge from the aeration devices (flocculent growth) rather than supernatant seed (dispersed growth) resulted in somewhat better 24-hr. B.O.D. reductions. This method was found to remove consistently more than 90 per cent of the B.O.D. after 24-hr. aeration (Figure 7). However, the actual reduction of B.O.D. by oxidation in the presence of

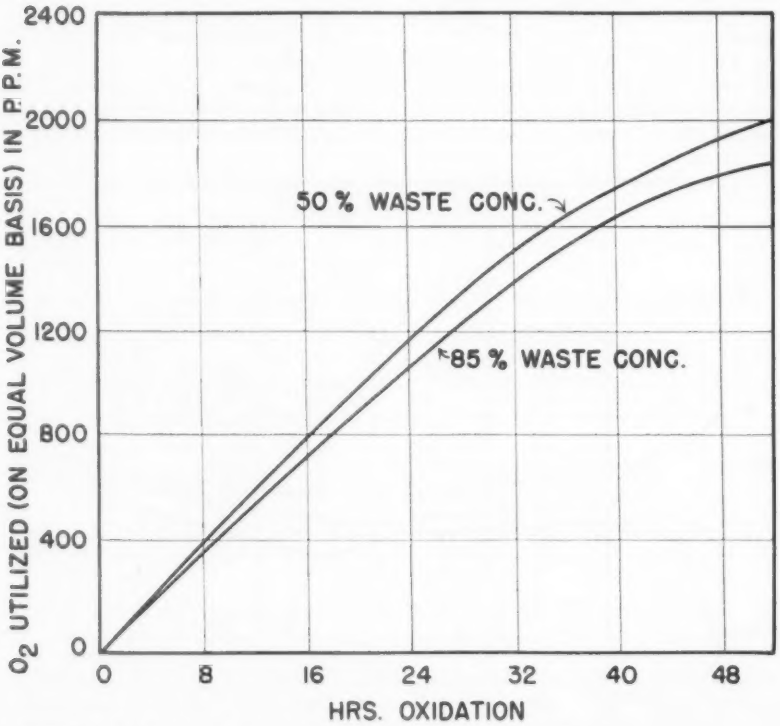

FIGURE 3.—Effect of increased neutralized, minerally fortified, rope waste on the oxidation rate of waste aerated with adapted seed.

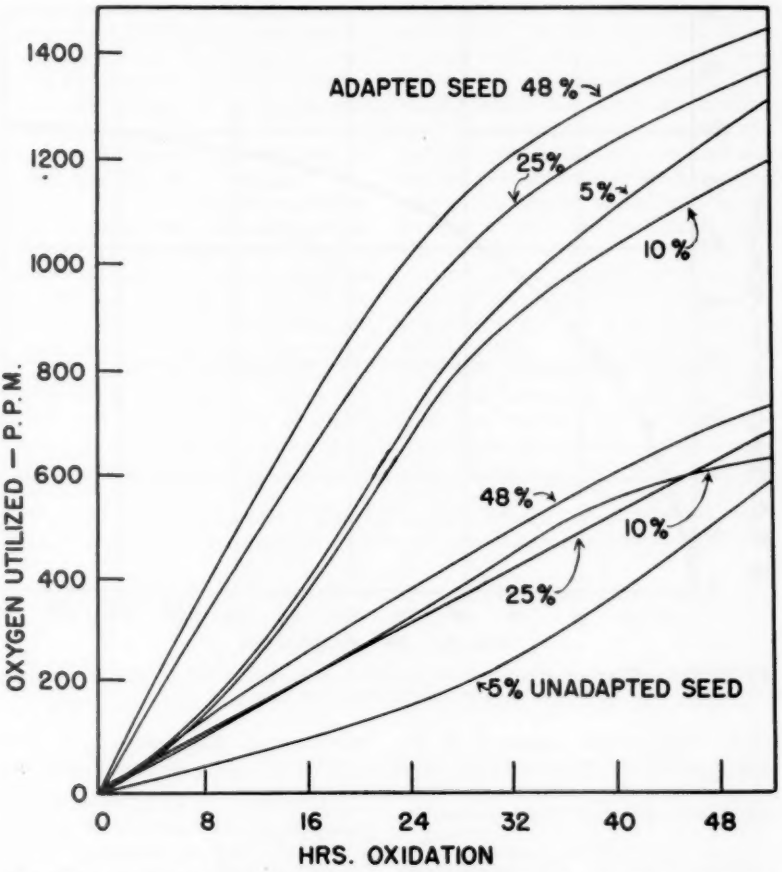

FIGURE 4.—Effect of seed adaptation on the oxidation rate of neutralized and nutrient (N and P) fortified rope waste.

sludge seeding appeared almost identical with that which occurred in the presence of dispersed-growth seed. The increase in the total B.O.D. reductions with sludge seeding appeared to be the result of adsorption of organic material on the sludge after the initial mixing of the sludge seed with the raw waste. A part of the adsorbed material was subsequently oxidized when the sludge was returned and reaerated. It is doubtful whether the additional B.O.D. reduction warrants the use of the flocculent-growth method when all the inherent difficulties of bulking, clogging, sludge disposal, and control are considered.

It is possible to decrease the total air consumption by reducing the time of aeration and, hence, the detention time of the waste in the aerator. Reduction in aeration time results in a lower B.O.D. removal, but not in direct proportion to the decrease in detention

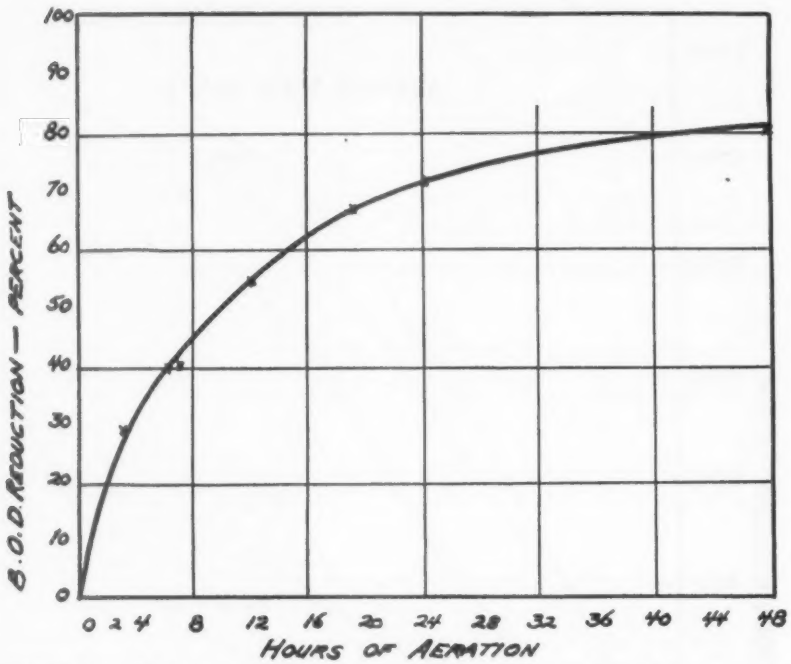

FIGURE 5.—Effect of aeration time on B.O.D. reduction at 20° C. and air supply of 2.75 cu. ft. per gallon per hour.

time. This is best expressed by the following values, which show the effect of short aeration periods at 35° C. on the B.O.D. reduction of rope waste:

Aeration Time (hr.)	B.O.D. Reduction (%)	
	Sludge Seeding	Dispersed-Growth Seeding
6	75	38
12	87	57
24	97	86

It is evident that with sludge seeding (flocculent growth) the aeration period can be reduced considerably without greatly affecting B.O.D. reduction. This is not so when the waste is aerated in the presence of dispersed-growth seed. It may be deduced that the sludge aids in bringing the food in closer contact with the organisms in a shorter time than the dispersed-growth seed, probably because of adsorption of the food on the sludge.

Aeration of Rag, Rope, and Jute Wastes Under Optimum Conditions

Experiments with rag, rope, and jute wastes liquors and wash-water mixtures in the presence of flocculent or dispersed seed with continuous daily addition for at least a week after the seed had been fully adapted showed that, in general, flocculent-growth (sludge) seeding was more effective than dispersed-growth seeding during the first 4 to 6 days of aeration (Figures 8, 9, and 10). After the first and succeeding aeration periods the sludge seeding became less effective than dispersed-growth seeding, probably because the absorbing capacity of the sludge was reached after 4 or more days and dispersion of some of the organic matter in the floc occurred. In other words, the rate of oxidation of the adsorbed

organic matter did not keep pace with the rate of adsorption of new organic substances. When the sludges were removed and aerated alone for a 24-hr. period, the activity of the sludge was renewed. Contrarily, the B.O.D. reduction with dispersed-growth seed increased with each daily transfer.

At the end of the eighth successive 24-hr. transfer with dispersed-growth aeration, the rag waste B.O.D. was reduced 92 per cent (Figure 8), the rope waste 78 per cent (Figure 9), and the jute waste 96 per cent (Figure 10), indicating that both rag and jute wastes are more amenable to biological treatment than is rope waste. When the sludge seed was kept fully activated with separate reaeration, corresponding B.O.D. reductions for sludge seeding were 93, 85, and 98 per cent.

Obviously, very high percentages of B.O.D. reduction can be obtained with both methods of aeration of jute and rag wastes. The rope waste B.O.D. reduction with dispersed growth was better than previously recorded, obviously because increased adaptation of the dispersed-growth seed took place. The increased adaptation occurred also without the addition of fresh sewage; hence, seed adaptation and activity can increase without the continuous addition of fresh sewage.

Only the dispersed-growth seed activity of the rope waste had reached its optimum at the end of the eighth consecutive 24-hr. aeration period. The other two seed materials (jute and rag) were still increasing in activity. Possibly they would prove as efficient as, if not more efficient than sludge seeding after a long adaptation period. In any event, aeration of rag, rope, and jute cooking liquors and wash-water waste mixtures in the presence of either dis-

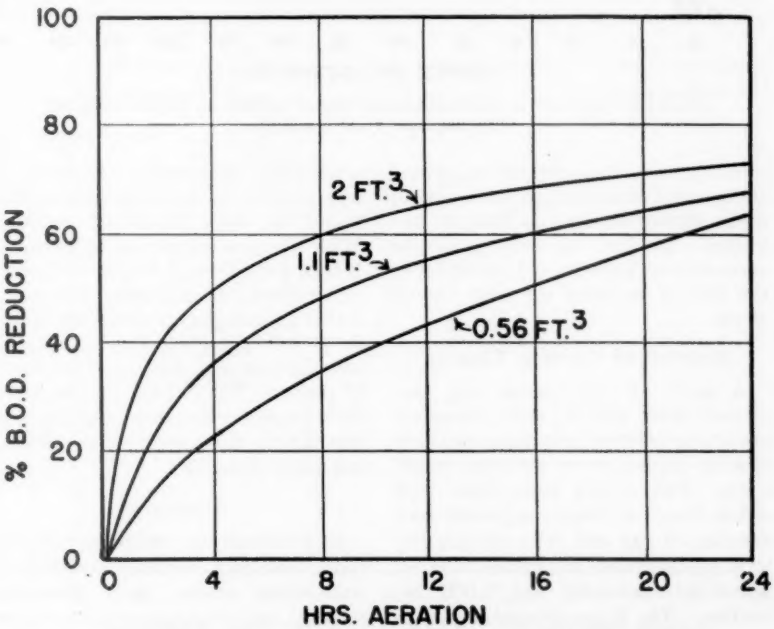

FIGURE 6.—Effect of air rate on B.O.D. reduction; rate of air in cubic feet per gallon per hour.

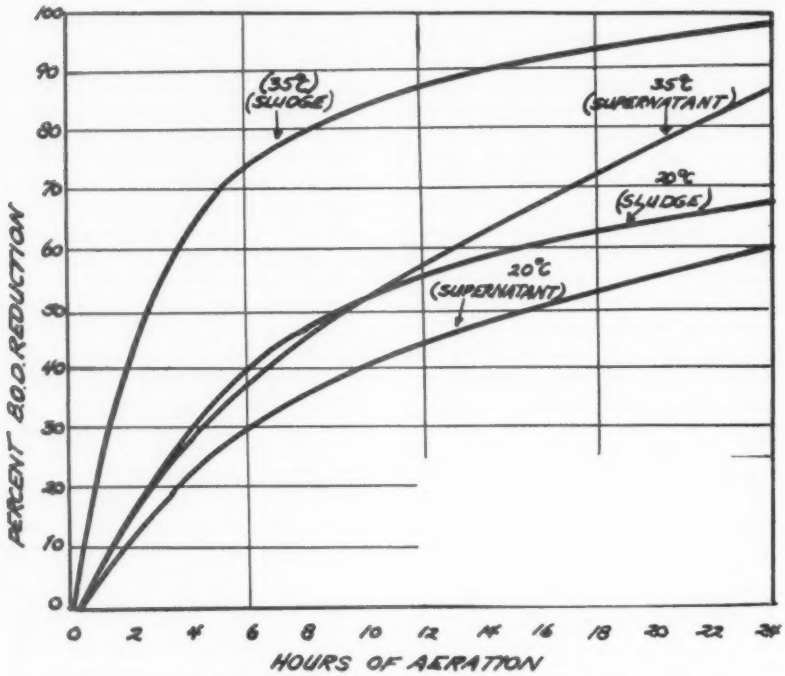

FIGURE 7.—Effect of temperature and type of seeding on B.O.D. reduction with dispersed-growth aeration.

persed-growth or separately reaerated sludge seed is capable of producing a high degree of stabilization of the wastes. In fact, in some cases the aerators were underloaded, which led to the idea of aerating the cook liquors alone.

Aeration of Cooking Liquors

A series of experiments was conducted with sludge and dispersed-growth aeration of rag, rope, and jute cooking liquors under optimum conditions. The results show that with either sludge or dispersed-growth seed aeration of rag and rope cooking liquors, a voluminous and stable foam resulted with relatively low B.O.D. reduction. The foam formation was so excessive that the aeration methods of the rag and rope cooking liquors were considered impractical. However, it was possible to treat the jute cooking liquor by dispersed-growth aeration. With an aeration period of 24 hr., a B.O.D. reduction of 85 per cent could be obtained, leaving about 480 p.p.m. B.O.D. in the effluent. Loadings of 0.15 lb. B.O.D. per cubic foot of aeration capacity per day were used to obtain 85 per cent B.O.D. reduction, or about three times the maximum loading with one-half the detention time required in anaerobic digestion.

Summary

Experiments on oxidation of rag, rope, and jute cooking liquors and wash-water wastes with flocculent (sludge) and dispersed growths under optimum conditions, and experiments related to the main factors affecting the

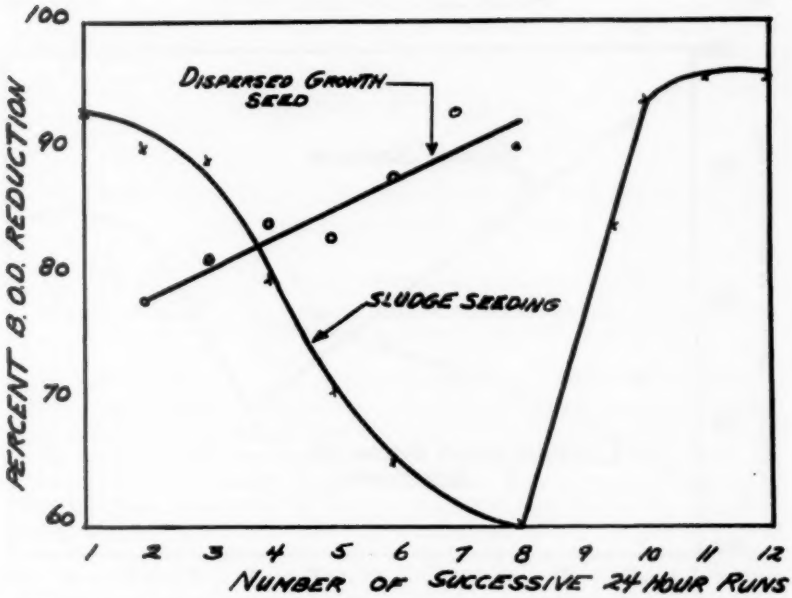

FIGURE 8.—Comparison of B.O.D. reduction with continuous aeration of rag waste in the presence of dispersed-growth and sludge seed.

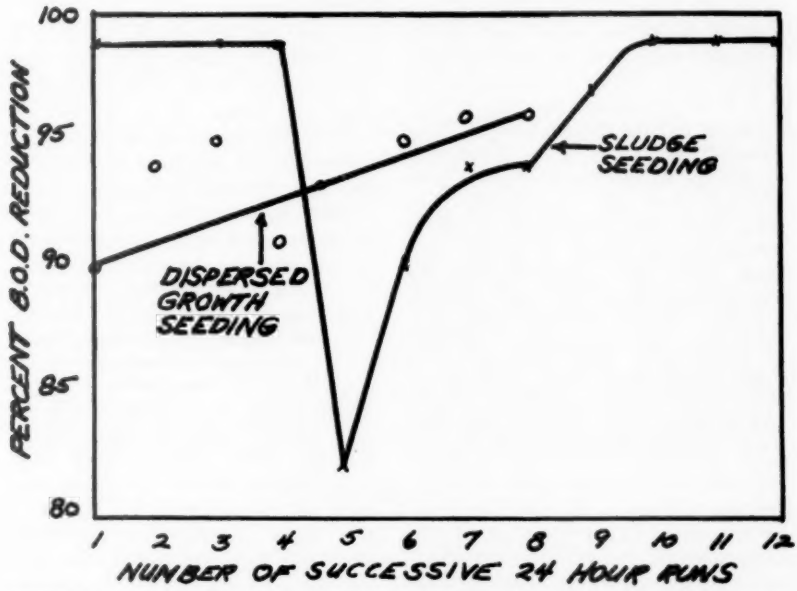

FIGURE 9.—Comparison of B.O.D. reduction with continuous aeration of rope waste in the presence of dispersed-growth and sludge seed.

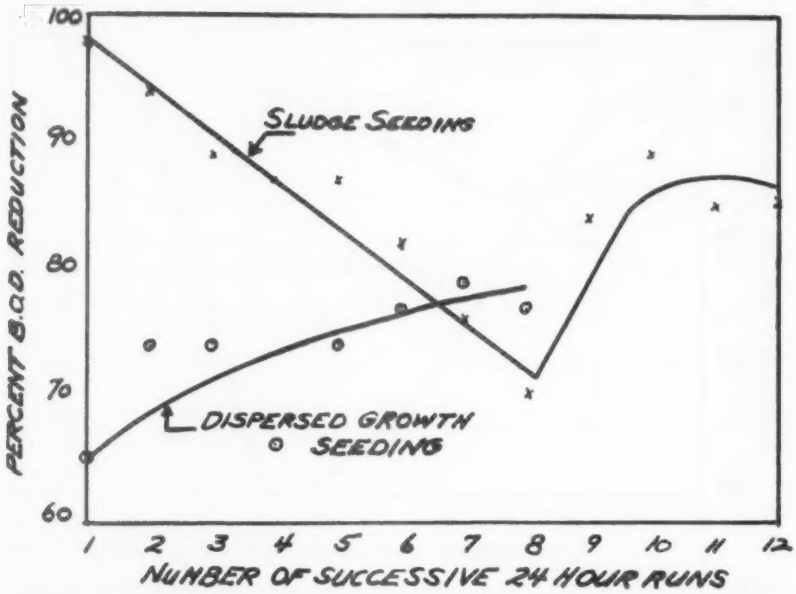

FIGURE 10.—Comparison of B.O.D. reduction with continuous aeration of jute waste in the presence of dispersed-growth and sludge seed.

rate of oxidation, show that aerobic biological methods of treatment of these wastes were effective in reducing the B.O.D. of the wastes. Aeration of mixtures of cooking liquors and the first hour of wash water in the presence of sludge floc appeared to be slightly more effective, but less dependable, than aeration in the presence of dispersed growth for reduction of B.O.D. The more important factors affecting the rate of oxidation of the wastes are pH, nutrients (N and P), seed adaptation, air supply, and temperature. At optimum conditions, B.O.D. reduction of mixed wastes with a B.O.D. concentration of about 2,000 p.p.m., in the presence of dispersed growth was 78 to 96 per cent, and with sludge floc 90 to 98 per cent, depending primarily upon adaptation of seed, after 24-hr. aeration. All mixed wastes and the jute cooking liquors could be treated by either type of aeration, whereas rope and rag cooking liquor could not be readily treated because of excessive voluminous and stable foam formation.

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Stream Pollution

QUANTITATIVE ESTIMATION OF *SALMONELLA* IN IRRIGATION WATER

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In a previous paper (1) the authors reported the isolation of naturally-occurring *Salmonella* organisms from 23 of 113 samples of sewage-contaminated irrigation water. Attempts to isolate pathogens from 45 samples of celery, lettuce, and cabbage, however, yielded no positives. The methods used in this preliminary study were not strictly quantitative, and as the practical application of this research depends on the numerical relationship of the pathogens to the common index organisms, coliforms and enterococci, attempts have been made to develop quantitative techniques for the isolation of *Salmonella*.

Methods and Results

Although *Salmonella* had been isolated on occasion from 1-, 10-, and 100-ml. inocula in the previous study, the 1-l. samples had consistently been negative. However, the methods used for handling the different volumes were not the same, and the Seitz pads employed with 100-ml. inocula were not satisfactory for large samples of 1 l. or more. Furthermore, using pure cultures of *Salmonella*, it appeared that many of the organisms were lost in the Seitz filtration and a large proportion were lost in the Foerst centrifugation. Finally, the efficiency of the tetrathionate broth-bismuth sulfite plate combination was not completely satisfactory.

To test further the loss of *Salmonella* in filtration through Seitz pads, a number of experiments were performed with pure cultures, employing disintegration and trituration of the pad and even reverse flushing with sterile water. In none of these was there recovery of more than 15 per cent of the organisms added. It would appear, therefore, that many of the organisms were killed or injured by the filtration, because even when the disintegrated pads were plated out the organisms did not grow.

Efforts were then made to apply chemical flocculation prior to centrifugation in a Foerst continuous centrifuge and in a conventional horizontal centrifuge. It was found that flocculation with ferric sulfate at a pH of 9.0 gave better recoveries of coliform organisms than either aluminum sulfate or ferric chloride. The aluminum sulfate even seemed to have a bactericidal effect on the organisms. Flocculation, however, even with ferric sulfate, was of much lesser value with the Foerst centrifuge than with the horizontal type.

Galton *et al.* (2) reported an increased recovery of *Salmonella* from dog feces through the addition of sodium sulfathiazole to Kauffmann's brilliant-green-tetrathionate broth, prepared by modifying Difco tetrathionate broth base (3). They also reported good results in streaking from this enrichment medium to brilliant-green agar plates. Accordingly, experiments were set up to compare the recovery of *Salmonella* from naturally-contami-

nated irrigation water with ordinary Difco tetrathionate broth and with Galton's modification, and at the same time to compare three plating media—brilliant-green, bismuth sulfite, and SS agars (Difco). The three plating media were streaked from the enrichment broths at 9, 18, and 24 hr. with the following conclusions:

1. The modified tetrathionate broth gave a greater number of isolations of *Salmonella* than did the ordinary tetrathionate broth.
2. More *Salmonella* were recovered from brilliant-green agar plates than from either bismuth sulfite or SS, the SS plates appearing to be the poorest of the three.
3. Contrary to experience in the 1950 investigations the 18- and 24-hr. incubations of enrichment broths were more productive than the 9-hr. period.
4. Centrifugation in the Foerst continuous centrifuge yielded just as many if not more pathogens than flocculation followed by centrifugation in the International centrifuge.

Up to this time the methods used in these studies for recovering *Salmonella* from water or vegetable washings had largely been semi-quantitative. It was now possible to give greater attention to the quantitative aspects of the methods. As the volume of sediment obtained from the continuous centrifuge could be related to the original volume of water used, the multiple-tube-dilution technique, in geometric series, was tried on the sediment. That is, three 1-ml. portions, three 0.1-ml., and three 0.01-ml. quantities of sediment were inoculated into tubes of Kauffmann's tetrathionate broth, single strength, as modified by Galton. At the same time, in many of the experiments, sediments obtained by flocculation with ferric

TABLE I.—Quantitative Determinations of *Salmonella* in Irrigation Water (Foerst and International Centrifuged Sediments Inoculated into Modified Tetrathionate Broth, 3 Tubes Each of 1.0, 0.1, and 0.01 Ml.)

Sample Number	Source	Quant. of Water Centrifuged (ml.)	Concentration Ratio	Type of Centrifuge	Incubation of TT Broth (hr.)	Number of Tubes Positive in Series		MPN per 100 Ml., Corrected for Conc.	
						Bril.-Green	Bism. Sulf.	Bril.-Green	Bism. Sulf.
397	So. Platte (York St.)	333	60:1	Foerst	18	3,0,0	3,0,0	4	4
				Internat.	18	2,0,0	1,0,0	1.5	0.6
400	So. Platte (York St.)	500	100:1	Foerst	20	2,0,0	0,0,0	0.9	<0.036
				Internat.	20	2,0,0	0,0,0	0.9	<0.036
401	Gardeners' Ditch	3,000	300:1	Foerst	18	3,3,1	3,0,0	15	7
				Internat.	18	2,2,0	0,0,0	7	<0.036
402	Gardeners' Ditch (flume)	1,000	100:1	Foerst	18	3,1,0	1,1,0	4.3	0.7
					24	3,1,0	1,1,0	4.3	0.7
				Internat.	18	3,1,0	0,1,0	4.3	0.3
					24	2,1,0	1,1,0	1.4	0.7
403	Gardeners' Ditch (Lammers)	1,000	100:1	Foerst	18	0,0,0	1,0,0	<0.036	0.36
					24	0,1,0	0,0,0	0.3	<0.036
				Internat.	18	0,0,0	0,0,0	<0.036	<0.036
					24	1,0,0	0,0,0	0.36	<0.036
405	Gardeners' Ditch (Clear Cr. overflow)	1,000	100:1	Foerst	18	0,0,0	0,0,0	<0.036	<0.036
					24	0,0,0	0,0,0	<0.036	<0.036
				Internat.	18	0,0,0	3,0,0	<0.036	2.3
					24	2,0,0	2,0,0	0.9	0.9

TABLE II.—Quantitative Determinations of *Salmonella* in Irrigation Water (Foerst Centrifuged Sediments Inoculated into Modified Tetrathionate Broth, 3 Tubes Each of 1.0, 0.1, and 0.01 Ml.)

Sample Number	Source	Quant. of Water Centrifuged (ml.)	Concentration Ratio	Type of Centrifuge	Incubation of TT Broth (hr.)	Number of Tubes Positive in Series		MPN per 100 Ml., Corrected for Conc.	
						Bril.-Green	Bism. Sulf.	Bril.-Green	Bism. Sulf.
407	Gardeners' Ditch (Lammers)	2,000	200:1	Foerst	18	3,1,0	3,2,0	2	4.5
					24	3,3,0	0,0,0	12	<0.036
409	Rocky Mt. Ditch (Bacher's)	2,000	200:1	Foerst	18	0,0,0	0,0,0	<0.036	<0.036
					24	0,0,0	0,0,0	<0.036	<0.036
411	Clear Cr. (Hughstons)	2,000	200:1	Foerst	18	0,0,0	0,0,0	<0.036	<0.036
					24	0,0,0	0,0,0	<0.036	<0.036
413	Petersburg Ditch (Nakamura's)	2,000	200:1	Foerst	18	0,0,0	0,0,0	<0.036	<0.036
					24	0,0,0	0,0,0	<0.036	<0.036
417	Gardeners' Ditch (Lammers)	2,000	200:1	Foerst	18	3,0,0	1,0,0	1	0.18
					24	2,0,0	3,0,0	0.45	1

sulfate and centrifuging in the International centrifuge (1,500 r.p.m. for 15 min. in size No. 2) were similarly inoculated into the enrichment medium. After 18- and 24-hr. incubation at 37° C., a large loopful of each enrichment growth was streaked on brilliant-green and bismuth sulfite agar plates, which were incubated for 20 to 24 hr. at 37° C. Representative colonies typical of *Salmonella* were picked to Christensen's urea agar (Difco) to eliminate *Proteus* and some of the paracolon organisms; cultures which showed no alkalinity on this medium within 24 hours were transferred to triple sugar iron agar. Final identification included the usual biochemical tests for *Salmonella* and agglutination with polyvalent *Salmonella* antiserum, according to the method of Hajna and Damon (4). All positive cultures were confirmed and identified as to species by the Communicable Disease Center, U.S.P.H.S., Chamblee, Ga.

Tables I and II show the results of these experiments. In general the technique worked quite well, the largest inoculum showing the greatest number of positives and the smallest inoculum, the least positives. By assuming that one *Salmonella* could produce a positive tube in order to use the most probable numbers tables (5), and correcting for concentration, quantitative data on the number of *Salmonella* per 100 ml. of original water were obtained. Eight out of 11 samples of irrigation water were positive for *Salmonella*. The median value for all 11 samples was 0.9 *Salmonella* per 100 ml. In the six experiments in which two types of centrifugation were compared, the Foerst gave higher counts of pathogens than the International in three experiments; the International was higher than the Foerst in one; and in two experiments the counts were essentially the same.

It may be concluded from these experiments that for practical purposes centrifugation in the Foerst is just as satisfactory as flocculation and centrifuging at 1,500 r.p.m. in the International. At the same time it is much faster and simpler to handle large quantities in the Foerst. Again, the brilliant-green agar in general gave more isolations than bismuth sulfite agar, although this was not always the

case. Finally, the application of the multiple-tube-dilution technique with the sediment was shown to be entirely feasible.

Table III shows the results of coliform and enterococcus counts on the samples just discussed. The median, high, and low values for both coliforms and enterococci were much higher in the 8 samples which were positive for *Salmonella* than the arithmetic averages and the high and low values obtained on the 3 *Salmonella* negative samples. The sample (No. 401) which gave the highest number of *Salmonella*, 15 per 100 ml., also showed the highest coliform and enterococci counts. Likewise, the sample which had the lowest coliform and enterococci counts was negative for *Salmonella* (No. 411). Ratios of coliforms to *Salmonella* and enterococci to *Salmonella* of 255,000:1 and 4,800:1, respectively, may be computed from the median values obtained from all 11 samples. Median values for coliforms and enterococci show a ratio of 54:1.

With the advent of the harvest season these techniques were applied to the washings from celery, cabbage, corrots, and onions picked up directly from the fields of various farms in the irrigated area. Plants were removed from the ground by placing a sterile paper bag over the entire plant and either twisting it from the roots or pulling the whole plant up without touching it directly with the hands. With sterile forceps and knife, 200 to 500 g. of the edible portion of each sample were transferred to a sterile screw-capped gallon jar. Sufficient sterile tap water was added to make a 1:10 mixture, which was allowed to stand 10 min. before being shaken vigorously 50 times. The washings were then centrifuged in the Foerst continuous centrifuge; the sediment was made up to a definite volume, and either 4 or 5 tubes of modified tetrathionate broth were inoculated, each with 1 ml. of the suspended sediment. The total quantity of sediment so inoculated in each case represented 83.3 g. of the original vegetable. The remaining sediment was used for coliform and enterococcus counts, and for animal parasite determinations. Of 14 samples of vegetables processed in this way only one was positive for *Salmonella*. This was a sample of celery from the first farm on Gardeners' Ditch. The calculated MPN value for this one sample was 1.32 *Salmonella* per 100 g. of vegetable.

Rudolfs (6) has emphasized the difficulty in removing coliform organisms from vegetables by simple washing and has suggested the alternative of blending the vegetables in a Waring blender. Therefore, we compared the recoveries of *Salmonella*, of coliforms, and of enterococci obtained by direct washing and by blending. In addition to the washing procedure just described, 100 g. of each of the same samples of vegetables were blended with 900 ml. of sterile tap water for 90 sec. in a Waring blender. This mixture was centrifuged in the continuous centrifuge and the sediment was made up to a definite volume. Again, 4 or 5 tubes

TABLE III.—Summary of Most Probable Numbers of Coliform Organisms and Enterococci in Irrigation Water, Samples 397–417 (MPN Counts in Thousands per 100 Ml.)

	All Samples		*Salmonella* Positive		*Salmonella* Negative	
	Coli	Entero	Coli	Entero	Coli	Entero
Median	230	4.3	230	19	34[1]	0.32[1]
High	>11,000	93	>11,000	93	43	0.43
Low	15	0.093	23	0.43	15	0.093

[1] Arithmetic average.

TABLE IV.—Comparison of Coliform and Enterococci Counts on Vegetables Using Direct Washing and Blending

Sample Number	Vegetable	Irrigation Source	Coliform Counts (MPN/100 g.)		Enterococci (MPN/100 g.)	
			Washing	Blending	Washing	Blending
404	Celery	Gardeners' Ditch	1,380,000	258,000	<2	84
406[1]	Celery	Gardeners' Ditch	558,000	138,000	1,380	5,580
408A	Cabbage	Gardeners' Ditch	1,150	<18	<2	<2
408B	Celery	Gardeners' Ditch	1,150,000	105,000	<2	<2
410	Celery and broccoli	Rocky Mt. Ditch	36	45	45	<2
412	Celery	Clear Creek	5,500	8,000	<2	<2
414	Carrots	Petersburg Ditch	115,000	4,650	180	<18
415	Onions	Petersburg Ditch	<2	<2	45	<2
418	Celery	Gardeners' Ditch	4,650	11,500	<2	<2
419	Celery	Gardeners' Ditch	18	11,500	<2	<2
421	Cabbage	Slough Ditch	<2	455	<2	<2
422	Celery	Slough Ditch	<2	1,150,000	<2	<2
425	Celery	Fisher's Ditch	>5,500,000	>5,500,000	<2	<2
426	Celery	Fisher's Ditch	11,500	115,000	<2	<2

[1] *Salmonella montevideo* isolated.

of modified tetrathionate broth were inoculated, each with 1 ml. of the suspended sediment, the quantity inoculated representing 83.3 g. of vegetable as before. The remaining sediment was used for coliform and enterococcus counts. The *Salmonella* obtained from the single positive vegetable sample were recovered from simple washings and no pathogens were recovered from the blended sample. This, however, may be just chance. Examination of the coliform and enterococcus data (Table IV) shows no consistent trend in favor of either direct washing or blending.

Because the portions of vegetables subjected to washing and the portions blended, although coming from the same original sample, could not be depended upon to be identically contaminated, a modification designed to eliminate this objection was tried later on 8 samples of cabbage artificially contaminated with a fecal suspension added to irrigation water and allowed to dry at room temperature for 24 hr. Five hundred grams of each sample were washed with 2 l. of sterile tap water. One liter of the washings was centrifuged and the sediment run for coliforms and enterococci. The remaining liter of washings and half the washed vegetables (250 g.) were blended, centrifuged, and the sediment likewise run for coliforms and enterococci. Again, no consistent trend could be demonstrated in favor of either washing or blending.

Discussion

Although the statistical calculations involved in the most probable numbers tables are based on the assumption that one organism will produce a positive tube, in biological techniques this is rarely if ever attained. It is important, then, to estimate the approximate number of organisms required to produce growth in a given medium or to produce a positive in a certain procedure. By using pure cultures of *Salmonella typhi-murium*, triturated with sterile feces and suspended in sterile tap water, it was estimated that a median value of 3 organisms is required to produce a positive tube by direct inoculation into modified tetrathionate broth and streaking on brilliant-green agar after 18- to 24-hr. incubation at

37° C. If the suspension is centrifuged in the Foerst continuous centrifuge and the sediment is inoculated, a median value of 22 organisms is required. This difference probably results from a number of factors: the efficiency of the centrifuge, the effect, if any, on the organisms of centrifuging at rather high speed (15,000 r.p.m.), and the effect on the enrichment medium of a relatively greater amount of organic matter in the sediment. At any rate, it seems probable that the quantitative values obtained from the water samples may be somewhat low.

The finding of a significant number of irrigation water samples positive for *Salmonella*, while the vegetables irrigated with this water do not show significant contamination, is in line with previous experience. Possible reasons for this have already been discussed (1).

Summary and Conclusions

A quantitative method has been developed for the estimation of *Salmonella* in sewage-contaminated irrigation water. Of 11 samples of irrigation water, 8 were positive for *Salmonella*. The median value for all 11 samples was 0.9 *Salmonella* per 100 ml. Only 1 of 14 samples of vegetables irrigated with this water was positive for these organisms.

Ratios of 255,000 coliforms and 4,800 enterococci to one *Salmonella* were computed from the median values obtained from the water samples. In comparing coliform and enterococcus counts from vegetables subjected to washing and blending, no consistent trend was shown in favor of either method.

Acknowledgment

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POLLUTION CONTROL OF THE RIO GRANDE IN NEW MEXICO*

By Robert P. Lowe

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The Rio Grande forms about 1,240 mi. of the international boundary between the United States and Mexico. Therefore, the various phases of its control are of definite concern to the two countries. Actual boundary problems are handled by the International Boundary and Water Commission, United States and Mexico. However, the Rio Grande traverses the entire length of the state of New Mexico before it becomes the international boundary in the vicinity of El Paso, Tex. The purpose of this paper, therefore, is to present the current situation on water pollution control in this drainage basin in New Mexico.

Basin Description

Historically the Rio Grande Valley in New Mexico is the oldest continuously settled area in the United States. Life in this area has been dependent on water as a limiting factor in the progress of development, and all water rights have been taken through basic water law derived from the appropriation doctrine of prior use. The demand for additional water has made the reuse of waters and the construction of reservoir storage projects important to continued prosperity in the basin.

The main stem of the Rio Grande and its major tributary, the Pecos River, have an all-inclusive basin that covers more than one-half of the 122,634 sq. mi. comprising New Mexico. However, as some of this area has closed desiccating basins with no connecting drainage to these rivers, actual tributary drainage is only from about one-third of the state. The Rio Grande originates in 14,000-ft. mountains in south-central Colorado, flows 170 mi. to the northern New Mexico state line, and thence traverses the state in a southerly direction for about 470 mi. Likewise, the Pecos River rises in 13,000-ft. mountains in north-central New Mexico, flows southward for approximately 425 mi. to the Texas state line, and continues 330 mi. across Texas for confluence with the Rio Grande.

Rainfall in the basin varies from as high as 35 in. per year in the northern mountains to less than 10 in. for a large portion of the drainage area. Runoff into Elephant Butte Reservoir, on the Rio Grande in south-central New Mexico, averages about 1,000,000 acre-ft. per year, and the discharge of the Pecos River at its mouth averaged 176,800 acre-ft. per year from 1924 to 1946. Reservoir storage capacity on the Rio Grande in Colorado and New Mexico totals close to 3,000,000 acre-ft., whereas the Pecos River has a total of nearly 500,000 acre-ft., of which Red Bluff reservoir, just on the Texas side of the state line, provides 307,000 acre-ft. These reservoirs regulate flow in the lower reaches of the state, so that dry stretches of river bed may result when no irrigation is in progress.

Elephant Butte Reservoir, with an estimated current storage capacity of 2,200,000 acre-ft., serves by far the largest irrigation project (Rio Grande Project, New Mexico-Texas), as 155,000 acres of land make use of this water.

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About 800,000 acre-ft. are required annually for irrigation and for provision of 60,000 acre-ft. to Mexico at El Paso as provided in the Mexican Treaty of 1906. Total irrigated land in the Pecos and Rio Grande basins in New Mexico is approximately 550,000 acres.

Three interstate river compacts control the use of water in these rivers. The Rio Grande Compact is administered by a commission composed of members from Colorado, New Mexico, Texas, and the Federal Government. The Pecos River Compact Commission is likewise administered by New Mexico, Texas, and the Federal Government. The third interstate agreement, the Costilla Creek Compact, is comparatively minor in nature, as it only involves use of water between Colorado and New Mexico on a minor tributary of the Rio Grande.

Water Uses

Irrigation of agricultural crops is by far the major use of water on these streams in New Mexico. Introduction of cotton as a crop has proved to be highly successful in the southern part of the state, whereas alfalfa, grains, and grazing lands are predominant users in the other areas. Municipal water supply is a minor use, as in most instances water is obtained from wells. Although watering of stock is also a comparatively minor use, it is important that minimum flows are available to this large activity in the state. Water shortages have definitely limited industries that require large amounts, so it has not been possible for them to become established in the state to any great extent. Recreational use of streams is quite important in upper mountain reaches and in reservoirs for fishing, but swimming and boating are very limited.

Water Shortage Problems

The extreme water shortage conditions of the past two years have made proper stream pollution control measures particularly desirable. From both the health and nuisance viewpoints, the situation is more serious when little or no dilution water is available. The draining of reservoirs has also created a problem in the disposal of dead fish, potentially a serious difficulty where large bodies of water are concerned. Although this situation fortunately did not arise at Elephant Butte, a federal court case arose from the situation that might result through complete drainage of this reservoir. Water shortages have accentuated any pollution problems and have required closer surveillance to alleviate or eliminate them.

Current Status of Control

Prior to 1945 only a few municipal sewage treatment plants existed in the Rio Grande and Pecos valleys of New Mexico, and the rapid growth of the state from a population of 531,818 in 1940 to 677,152 in 1950 created an imperative need for many new plants. This is particularly evident from the increase in urban population that is reflected in the last 10-year census growth of Albuquerque from 35,499 to 97,012 persons, of Roswell from 13,482 to 25,572, and Carlsbad from 7,116 to 17,975. Educational work by the New Mexico Department of Public Health and the initiation of post-war construction projects made a definite improvement in these conditions. Also, the entrance of the Federal Government into water pollution control activities by passage of Public Law 845, 80th Congress, has aided New Mexico in obtaining additional personnel, laboratory equipment, and closer coordination with the U. S. Public Health Service.

Water pollution in New Mexico may be broadly classified as municipal wastes, industrial wastes, silt from soil erosion, and dissolved salts. The first two contaminants can in turn be classified as man-made, whereas the last two are commonly called natural pollutants.

The main channel of the Rio Grande in New Mexico is the ultimate drainage course for sewage effluent from about

160,000 persons; actually there is normally little chance for more than 130,000 persons to contribute to this pollution, as some of the effluent is used for purposes that prevent direct return to the river. Of the total of 13 sewered communities that may contribute municipal wastes to the Rio Grande, 12 have provided new or enlarged facilities since 1946. All except two of these treatment plants have secondary biological treatment. The two exceptions have a total sewered population of less than 1,800, and even they are provided with primary treatment that is considered to be satisfactory.

The Pecos River in New Mexico receives the treated sewage effluent from approximately 75,000 persons, but some of this is also normally used for irrigation so that it does not always discharge direct to the stream. Of the ten communities whose wastes reach the river, five have provided new or enlarged treatment plants in the past three years and two of the remaining sources are now considering enlargement and rehabilitation of their facilities. One small community (less than 1,000 persons connected to the sewers) is still discharging raw sewage to the river, but preliminary steps already have been taken toward elimination of this condition. All but one of the nine sewage treatment plants that discharge effluent to the Pecos River are provided with secondary biological treatment, and this one exception has less than 900 sewered inhabitants.

In the past, industrial waste pollution problems have not been great in the state, as a large percentage of these wastes are connected to the municipal sewerage systems. These include milk pasteurization and dairy plants, slaughter and packing houses, laundries, small food processing plants, and various other small industries. However, one gasoline refinery, two cotton delinting plants, and one large packing house have contributed partly-treated wastes for discharge to the Rio Grande.

The gasoline refinery mentioned has recently been eliminated as a pollution problem, as all liquid waste is now pumped to ponds for disposal by seepage and evaporation. Elimination of this situation has minimized severe taste and odor problems that formerly existed in the El Paso, Tex., water supply. In addition, water diverted at the Mexican Dam at El Paso for use in Mexico has also been improved in quality.

A survey of the cotton delinting plants near Las Cruces, N. Mex., has revealed no particular problem from the discharge of dilute acid waste to a drainage ditch, as a considerable volume of water accumulates in the ditch prior to final discharge to the Rio Grande. Pretreatment of wastes at the large packing house near Albuquerque has prevented the creation of a problem from the discharge of this effluent to a large drainage ditch.

The milling of radioactive ores and the production of radioisotopes in the Rio Grande basin is becoming a more important problem. Disposal of radioisotopes has received very close supervision in the past and research has made improved methods available for treatment prior to final disposal. Milling of radioactive ores has only recently become established in the Rio Grande drainage area, and this development is being watched closely.

Industrial wastes that do not reach the municipal sewers are minor in the Pecos River basin in New Mexico. The escape of gasoline refinery wastes from two adjacent plants to the river on several occasions has resulted in comparatively local fish kill problems, but recent cooperation with these refineries has minimized the possibility of recurrence of such situations.

Both the Rio Grande and Pecos River carry large silt loads, as heavy spring runoffs from melting snows and sudden summer thunderstorms result in severe erosion of soil. Only an average of 12,800 tons of silt is carried in the Rio Grande annually at the Colorado-New Mexico state line, but 37,000,-

000 tons is picked up in New Mexico in the drainage area above Elephant Butte Reservoir. About 25,000,000 tons of this silt is deposited annually in the main river bed, and most of the remaining 12,000,000 tons is deposited in the quiet waters of Elephant Butte. Measurements have indicated that one-fifth of the capacity of this reservoir has been lost through silting in the first 32 years of operation. The capacity of McMillan Reservoir on the Pecos River has also been seriously diminished by silting.

The Pecos River has a dissolved solids content of over 6,000 p.p.m. at its entrance into Red Bluff Reservoir on the Texas side of the state line. Much of this is due to natural salt deposit drainage that exists in the immediate upstream area in New Mexico, as surveys have indicated a sharp rise in the content in this part of the river. The U. S. Geological Survey and the New Mexico State Engineer have made an extensive survey of this situation and recommendations have been made for alleviation of the condition. However, the cultivation of salt-resistant crops, such as cotton, in the valley below has not created a demand for action so far. A comparison with the Rio Grande, with its much larger drainage area and a greater discharge, shows that at the confluence with the Pecos River it carries only about 1,300,000 tons of dissolved solids annually, in contrast to the 1,600,000 tons from the Pecos.

Future Outlook

The demand for use of all available water in New Mexico, Texas, and the Republic of Mexico necessitates close control of pollution so that the maximum usage of water may be obtained. This need has already been reflected in New Mexico through the constant demands for use of sewage effluents for irrigation. Growth of the Southwest can be expected to continue as it has in recent years, and this will result in the need for additional domestic sewage and industrial waste treatment facilities in the future. The development of the atomic energy industry, and the growth and introduction of other industries, can be expected to create some additional waste problems. However, continued and alert control can prevent extensive problems from being created.

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THE OPERATOR'S CORNER

CONDUCTED BY HERBERT P. ORLAND

FORESIGHT PAYS OFF

Early in 1949, when Kerwin L. Mick, chief engineer and superintendent of the Minneapolis-St. Paul Sanitary District, presented to his Board of Trustees a flood emergency program, he did so with the statement that it "probably never will be needed." Nevertheless, his foresight in having the program prepared was to be vindicated in April, 1951, when the flooding Mississippi River almost topped the plant's protective dike, and again in April, 1952, when the dike was topped by the largest flood on record and some 4 ft. of water covered the entire plant area.

Details of the program, aimed at sealing the lower levels of the plant buildings and all the connecting tunnels, were prepared by B. M. Storey, assistant engineer, and are given elsewhere in this "Corner." The effectiveness of the planning and preparation is attested by the fact that almost all of the $5,000 damage caused by the flood was done to plant grounds and landscaping features, whereas the potential damage to plant buildings and equipment below the flood level was measured in millions of dollars. Private industries in the area suffered many times the amount of damage sustained by the sewage treatment plant, mostly due to lack of prepared flood emergency programs.

After a personal inspection of the plant during the flood's crest, the chairman and the secretary of the District's Board of Trustees presented to the entire Board a communication describing the conditions as they found them. In this report they said:

"We believe that a great deal of credit is due Kerwin L. Mick, the District's chief engineer, and the plant personnel, for the foresight and action taken to protect the sewage treatment plant from serious damage during the present disastrous flood. . . .

"When one sees what could have happened to the entire plant had not proper precautions been taken in ample time, one feels very grateful to the plant personnel. If the river had succeeded in breaking in faster than it could have been pumped out, and the various basements and tunnels flooded, there is no doubt that extensive damage would have resulted to the equipment located therein. It is estimated that at present-day prices the value of equipment in the plant, much of which would have been submerged, is slightly in excess of $3,000,000. We believe it is fitting, therefore, that we, as a Board of Trustees, go on record as commending Mr. Mick and the plant personnel for the magnificent job they have done."

With this go also, we feel sure, the heartfelt commendations of fellow superintendents and operators, who recognize another example of unselfish devotion to duty in public employ in time of emergency. In addition, the "probably-never-will-be-needed" aspect of the situation was not allowed to defer completion of a plan of action, so that it was ready at a moment's notice when needed and the required materials and special fittings were on hand to do the job.

Has your plant, faced with the same potential emergency situation, a similarly detailed and up-to-date plan of action?

H. P. O.

ADVANCE PLANNING CUTS FLOOD LOSS

By Kerwin L. Mick and Benjamin M. Storey

Chief Engineer and Assistant Engineer, respectively, Minneapolis-St. Paul (Minn.) Sanitary District

When the Mississippi River at St. Paul, Minn., during April, 1952, reached the highest flood stage in 85 years of record, only detailed advance planning and painstaking preparation saved the sewage treatment plant of the Minneapolis-St. Paul Sanitary District from extensive flood damage. Actually, the major part of the planning had been done early in 1949, when a flood emergency program had been prepared. Revisions were then made from time to time to keep the program up to date. In fact, the latest revision was made as recently as January, 1952.

To appreciate the complexity of the planning involved, it is necessary to understand the conditions existing at the plant. The plant is located at Pig's Eye Island, downstream from St. Paul, with the plant grounds generally at elevation 700 ft. In an 85-yr. period of record, the river level at St. Paul has exceeded that elevation on 12 occasions. However, the plant's settling tanks are not operative after the river elevation reaches 698. The highest river level on record at St. Paul previous to preparation of the plan was elevation 704 and occurred in April, 1881. It was on this basis that the plant dike had been constructed with a top elevation of 703, which again was nearly reached by the river in April, 1951.

The elevation of the first floor level in the main buildings at the plant is 706.5. Floors in the tunnels connecting the various buildings are at elevation 682, with the incinerator room basement floor at elevation 676.

Flood Emergency Program

Principally, it was at sealing off the basements and connecting tunnels and keeping them dewatered that the flood emergency program was aimed. Included in the records of the scheme, all carefully kept up to date and on file for immediate use, was a plan showing the areas to be protected, a detailed schedule of the proposed protective measures for each area, a complete directory of local and governmental flood agencies and personnel, and a copy of the "Flood Emergency Manual" published by the U. S. Army Engineers.

Major sections of the detailed emergency program are quoted as follows:

Working Conditions

It is possible that Childs and Warner Roads would be submerged, preventing entrance to the plant by automobile. In that event the nearest point from which the plant could be reached by car would be Daytons Bluff or Fish Hatchery Road. From there it would be necessary to wade across the railroad tracks and walk in over the top of the interceptor sewer barrel. River transportation may also be considered.

Maintaining a certain crew at the plant might be more logical, in which case it would be necessary to provide sleeping accommodations, such as cots, etc., and food supplies. Meat, bread, and other perishable food could, no doubt, be transported to the plant by the Coast Guard, who will be patrolling the river.

Intercepting Sewer

By-pass sewage flow at 3rd and Commercial Streets and at the plant.

Consider by-passing Minneapolis flow.

Screen and Grit Building

Plug all floor drains on the track slab and in the garage with oakum. Then

weight down the covers and brace if necessary.

Seal the top of the manhole south of the building and over the tunnel running to the concentration tank.

Remove hose, lawn equipment, etc., from the track slab and the garage.

Consider the advisability of raising up trucks on blocking or storing some of the trucks uptown.

Pump and Blower Building

It is doubtful that any protection is needed for this building, unless possibly from wave wash and debris. In the event of this, or higher water, caulk and seal all doors, placing sandbags around the north entrance.

Venturi Meters

Seal the manhole covers over both vaults; also the pipes through the manholes.

River Sampling Boat

Make the necessary arrangements for docking the boat.

Settling Tanks

Seal the tops of the sidewalk covers at the east and west ends of the influent channel.

Close all valves on the underdrain system.

P–21 (Yard Drainage Pump over Outfall)

Brace the cover on the wet well.

Seal the sidewalk covers.

Effluent Service Building

Caulk the rear doors (bottom and approximately 4 ft. up on the sides).

Administration Building

Plug the floor drains throughout with oakum and weight down the covers; brace if necessary.

Plug the vent to the tunnel at the southeast corner.

Plug the pipe used for tank readings in the road.

Plug the oil tank filler opening and vent, or extend the vent.

Seal the top of the manhole to the boiler room with roof tar.

Caulk the rear doors (bottom and approximately 3 ft. up on the sides).

Sand bag the basement (garage) windows on the west and east sides.

Set a wood panel, caulking up same, in the window frame on the fresh air inlet, south side.

Caulk all around the garage door (overhead rolling type) and brace.

Close up the opening to the garage by laying up concrete blocks or brick in the door frame, leaving sufficient room on the sides and top to pack oakum and seal. Waterproof the exterior side (see "Material List").

Set wood panels, caulking up same in the openings for the locker room fan and the lunch room vent.

Filtration and Incineration Building

Install sandbags around:

Windows on the south side of the incinerators.
Windows on the east side of the incinerators.
Doors on the east side of the incinerators.
Areaway around the boiler room stairwell.

Caulk and seal:

Windows on the south side of the incinerators.
Windows on the east side of the incinerators.
Door on the east side of the incinerators.
Windows in the boiler room areaway.
Door in the boiler room areaway.

Extend the following so that they are approximately 3 ft. above ground:

Fill pipe for the gasoline storage tank.
Vent from the boiler make-up tank.
Vent from the boiler blowoff basin.

Caulk and seal around the oil unloading line at the east wall.

Seal the floor drain in the boiler room areaway.

Plug the steam and oil unloading lines at the railroad track.

Heating

Have ample fuel oil on hand.

Concentration Tanks

Seal the sidewalk covers with roofing pitch.

$FeCl_3$ Storage Tank

Seal the manhole cover and around the pipe inlets.

Transformers, General Power, and Electrical Protection

The Northern States Power Company has a flood emergency program whereby it will be in a position to continue to supply power. Water can rise to elevation 707.0 on the power company transformers at the plant and they can still be in service. In the event service through these transformers is interrupted, power would be available through the 4,000-v. line.

Discontinue power through the transformers at the effluent service building and feed through the 220-v. tie, sealing up the 4-in. conduit on the transformer slab.

Seal the conduits in the pull box at the effluent service building.

Remove the motors, starters, and clutch coils on all effluent gates. Bolt on boards with gaskets where motors are removed, sealing up the conduits.

Remove the motors with reducers on all settling tank sludge collectors, except at the flocculating tanks, sealing up the conduits.

Seal the necessary conduits in the pull box at the main switchboard.

Seal the conduits on the ground flood and street lights.

Emergency Power

Extend the gasoline fill pipes on the tanks at the screen and grit building and at the filtration and incineration building.

Fill all the underground gasoline tanks and have several extra drums of gasoline on hand, arranging for suitable means of dispensing gasoline to underground tanks, providing cans, funnels, etc.

Check the gasoline engine for performance.

Telephone

The Northwestern Bell Telephone Company expects to have no interruption of its service. The telephone cables entering the buildings through conduits have been sealed off.

Auxiliary Pumping Equipment

Considerable additional seepage will enter the tunnels, principally through expansion joints, pull boxes, and cracks in the tunnel floors, the boiler room floor, and possibly other places which have not shown up to date.

Probably the most critical area where absolute drainage must be maintained is the basement of the pump and blower building. The following pumps are available for that purpose:

P12—300 g.p.m.
P13—300 g.p.m.
P14—200 g.p.m.
P15—200 g.p.m.
P 9—500 g.p.m.
P10—500 g.p.m.
P11—2,500 g.p.m. (part capacity)
P1–P8—200 g.p.m. each

Discharge from the pumps is to go to the common channel and through the sludge emergency line.

To help out in this area the following pumps are available:

P16—300 g.p.m., effluent service building.
P17—300 g.p.m., effluent service building.
P25—200 g.p.m., screen and grit building.

P34—200 g.p.m., concentration tank gallery.
P35—200 g.p.m., concentration tank gallery.

It appears that sufficient pump capacity would be available, but there might be a shortage of suction lines from the sumps in the pump and blower building, in which event some lines would have to be disconnected and temporary lines run to the sumps to obtain the additional capacity needed.

In the incinerator basement it would seem that the sump pumps, together with the ash sluicing pump, would take care of any additional seepage which might occur in that area.

NOTE: Install the special section with valve in the ash discharge line on the floor at elevation 688.08 (the section hangs on the east wall).

Sandbagging

The recommended practice is to use burlap bags or some other soft material that will readily conform to odd shapes. Sewing the top of the sacks is preferred for better fitting, but tying is also used. Fill the bags only one-half full. A sack rack can be built for filling bags. The sacks should be placed with joints broken, in parallel rows, with earth (or in our case sludge cake), used as a mortar between them.

As it is almost impossible to sandbag without some small amount of seepage, further precaution is to be taken by caulking and sealing around windows and doors, the sandbagging being a protection against wave wash and debris. It is estimated that it will take approximately 1,200 sandbags. Tarpaulins placed between the bags and the door or window are also desirable.

Material to Have on Hand

Roofing pitch
Waterproofing
1,400 burlap bags
Twine (if bags are sewed)
Needles (if bags are sewed)
Wire ties (if bags are tied)
Racks for filling bags
Shovels
Tarpaulins
2,000 common brick
6 sacks Portland cement
8 sacks mason's cement
Boots
Conduit sealing compound
Lumber
Oakum
Lead wool
Cotton
Permatex
Hose, pipe, and fittings for pump connections

Miscellaneous

Put solid covers on the catch basins near the chlorine contact tanks, then seal and cover them with sandbags.

Cover the grating over the outfall at the rear of the effluent service building with large oil-soaked canvas, then cover with sandbags. The covers are stored in the effluent service building.

Plug the outlet of the drain north of the flagpole as the river approaches elevation 697 (the outlet is on the riprap slope).

Curves are on hand showing the liquid level in test holes throughout the settling tank areas during high water for the year 1951.

1952 Flood Conditions

The maximum river level permitting plant operation (elevation 698) was exceeded on April 9, 1952. The sewage flow was then by-passed to the river above the plant and the flood control program was placed in effect. This was the flood emergency program, which had been submitted to the District Board of Trustees on Febraury 28, 1949, the major sections of which have been previously given. It is aimed principally at sealing off the basements and tunnels, and keeping them dewatered.

The plan had been given a "dry run" during the 1951 flood and some of the "kinks" worked out at that time. The water didn't get quite high enough then, however, to flood the plant grounds.

Doors, windows, manholes, vents, and filler pipes were sealed, plugged, or extended above predicted flood levels. Motors on the basement level were moved to a higher floor and additional pumps were installed to remove seepage water. Records in the basement vault were moved to upper levels in the administration building.

Most of the windows and vents were caulked, sandbagged, and even sealed with hot roofing asphalt. However, the big 8-ft. wide door on the basement level of the administration building was closed with a brick wall three bricks thick and reinforced with steel (see Figure 2). Every opening had to be closed tightly, as a major leak anywhere would have flooded the whole plant through the connecting tunnels.

The flood control program was substantially completed on April 10. The Childs Road approach to the plant went under water on the morning of April 12 (Figure 1),* and that night the river began flooding the plant grounds. By Easter Sunday morning, April 13, the grounds were entirely flooded. Early that morning there was a cave-in of the ground along the south wall of the screen and grit chamber

FIGURE 1.—Aerial photo of Minneapolis-St. Paul sewage treatment plant on April 17, 1952, after river topped dike and flooded plant. River, here seen at elevation 704.04, crested on April 16 at 704.24. River channel is to the right of the plant.

* Figure 1 courtesy of Kenneth M. Wright Studios. Others by St. Paul *Dispatch Pioneer Press.*

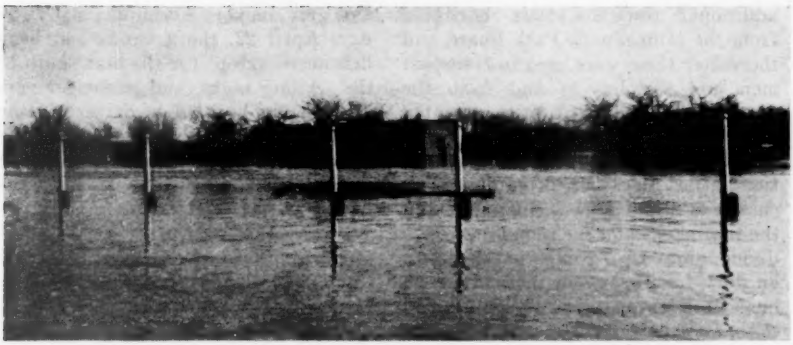

FIGURE 2.—View across inundated settling tanks from administration building. Water was over top of tank railings and half-way up on doors of effluent service building (center background).

building, the dirt washing down under a shallow curtain wall. The resultant rush of water blew out the caulking that had been placed around some electrical conduits leading from that building into a small tunnel into the concentration tank gallery. This proved to be the largest leakage of water into the tunnel-basement system experienced during the flood. Additional pumps were placed in service and were able to keep ahead of the infiltration.

On Sunday and Monday, with only two rowboats available at the plant, the men were rowed across from the Reserve Street Bridge to the top of the interceptor sewer box section, along which they then walked the distance of about 1 mi. to the plant.

On Tuesday morning, April 15, six

FIGURE 3.—Submerged ramp and bricked-up entrance (below light bracket) to garage in administration building basement. Picture taken with water 3 in. below crest level.

additional rowboats were borrowed from the Minneapolis Park Board and thereafter these were used to transport men and supplies to and from the plant, using outboard motors rented from employees.

To avoid the hazards of transportation after dark, the regular shift change at midnight was eliminated and the four shifts and the shop crews were divided into two groups—each group on duty for 24 hr. and then off for 24 hr. Each group was further divided so that one-half of the men at the plant were at their stations for 6 hr. and then rested for 6 hr., subject to call, however, in the event of an emergency. Cots and blankets were provided and a temporary commissary was operated in the laboratory to provide meals for the men. Men having special training or qualifications were assigned duties requiring that kind of training. Manpower was conserved so that fresh and alert men were always available.

On Wednesday, April 16, the river level at the plant crested at elevation 704.2, approximately 4 ft. above the average level of the plant grounds (Figures 2 and 3) and about 1½ ft. above the plant dike. This crest was approximately 2½ ft. above the previous high-water record, established in 1881.

Although the head of water on the basement floors and tunnels reached as much as 28 ft., the control program functioned successfully to prevent flooding of these areas. If the water had broken in, it would have been as much as 28 ft. deep around the incinerators and several million dollars worth of equipment would have been flooded.

By Sunday, April 20, the water had receded to the point where trucks could be used to drive in as far as the screen and grit chamber building. By Tuesday, April 22, the grounds had been dewatered except for the area south of the settling tanks, and passenger cars could again be used to drive into the plant. The regular shift changes were then resumed and the work of removing the sandbags and cleaning up after the flood was started.

On April 26, the river had receded to elevation 698, at which point operation of the plant theoretically could be resumed. It was necessary, however, to dry out the settling tank motors and do other work incidental to returning the plant to operating condition. Also, repair work on the lining of the incinerator chimney was started on April 23 (this work was not related to the flood) and continued through the balance of the month, the plant remaining shut down. The staff was occupied in clean-up work after the flood, removing deposits from the sludge and filtrate lines, cleaning and checking the venturi meters, tubes, etc.

Damage to the plant as a result of this flood was confined essentially to items of a minor nature, principally to the grounds in the form of washouts and from ½ in. to 1 in. of silt deposited on the grounds, roads, and walks. The total damage amounted to about $5,000, although some $18,700 was spent in the flood emergency program, mostly in overtime payments ($16,000) to the personnel.

The personnel of the District staff are to be commended for their fine spirit of cooperation and willingness to work hard and long hours, accomplishing the necessary flood protection work in a short period of time. Considerable information and valuable experience were gained which will be useful in dealing with future floods.

COMPOSITION OF AERATION TANK FROTH

By Wellington Donaldson

Deputy Director, Division of Sewage Disposal, Department of Public Works, City of New York, N. Y.

Despite the frequency of the question, "What is this froth that forms on aeration tanks?", few recorded analyses of actual froth content are available. A fair answer to the question might be: "Froth is mostly air, containing some liquid. The liquid contains a small amount of solids and the solids contain considerable grease."

As an example, to obtain 1 qt. of liquid for laboratory analysis it was necessary to condense (quite a trick in itself) about 9 cu. ft. of fresh froth from the aeration tank. The greasy, dirty liquid so obtained contained, by weight, 0.77 per cent total solids, 0.18 per cent grease, and 0.09 per cent alcohol soluble detergent. It is noteworthy that in this particular sample the grease was about 23 per cent of the total solids, detergent about 12 per cent of the total solids. The remainder of the total solids comprised sewage solids and activated sludge.

Froths derived from final tanks or from the final effluent of activated sludge plants contained a much higher percentage of grease, say from 30 to 50 per cent. Some foam accumulation dried on the sides of final tanks contained up to 90 per cent grease.

As an indication of the extent of the problem, Figures 1, 2, and 3 show conditions at New York City's 26th Ward plant during the summer of 1951. This plant is of the high-rate activated sludge type. Since the photographs

FIGURE 1.—Typical occurrence of froth on Aeration Tank at 26th Ward, without controls. Froth increases in volume with passage of sewage through four channels of the aeration tank in series from left to right.

FIGURE 2.—The "icebergs" shown are froth extruded from manholes on the effluent conduit of final settling tanks.

FIGURE 3.—This was a tough day for the sewage treatment worker who was attempting to collect final effluent samples from a manhole on the outfall conduit.

were taken, satisfactory control of the unusually severe frothing condition at this plant has been provided by a permanent system of spray piping and equipment for injecting a controlled dose of one of several available "anti-foams" into the spray system when and where needed. Neither the spray nor an anti-foam chemical alone has been found satisfactory, but the combination appears to provide a complete and economical control.

SUPPLEMENTAL REPORT FOR MANUAL OF PRACTICE NO. 1, "OCCUPATIONAL HAZARDS IN THE OPERATION OF SEWAGE WORKS"

Resuscitation

On page 43 of Manual of Practice No. 1 under "Resuscitation" it is stated: "Resuscitation following gas collapse, drowning or electric shock should be employed *immediately* by the Schaefer prone-pressure method of artificial respiration and a doctor should be called at once."

Recently (1951), controlled experimental studies have indicated the superiority of "push-pull" methods of manual artificial respiration. Push-pull methods actively induce both inspiration and expiration, and are more than two times as efficient for pulmonary ventilation as the Schaefer prone-pressure (push only) method.

Of the "push-pull" methods, the back pressure-arm lift (Nielsen) method has been endorsed by the Council on Physical Medicine and Rehabilitation of the American Medical Association, the American National Red Cross, U. S. Department of Defense, U. S. Army, Navy, and Air Forces, U. S. Public Health Service, and many other organizations. This method is simple to learn, easy to apply, and will not unduly tax the strength of the operator. If one or both arms of the victim are broken, the older prone-pressure method may be used or, preferably, one of the hip-action back-pressure methods.

Procedure

General

Place the victim in fresh air in the face-down (prone) position. Bend both his elbows and place one of his hands on the other. Turn his face to one side and place it on his hands.

Quickly sweep your fingers through his mouth to clear out froth and debris and bring his tongue forward.

Rescue measures should begin at once and continue rhythmically until natural breathing is established—this may take four or more hours. Efforts to restore natural breathing should not cease until the body stiffens.

Steps

First—Kneel on one knee at the patient's head, facing his feet. Grasp both the patient's arms just above the elbows (see Figure 1).* If it is more comfortable, kneel on both knees, one on either side of the victim's head.

Second—Lift the arms upward while you rock backwards, drawing the arms toward yourself until you meet firm resistance (see Figure 2). When doing this, do not bend your elbows; keep your arms straight. The arm lift pulls on the victim's chest muscles, arches his back, and relieves the weight on

* Figures reproduced through the courtesy of the Connecticut State Department of Health.

FIGURE 1.

his chest; air is thus sucked into his lungs.

Third—Replace the patient's arms on the floor and slide your hands to his mid back, with one hand just below each shoulder blade and the fingers pointing down (see Figure 3).

Fourth—Rock forward, exerting pressure almost vertically down to cause active expiration (see Figure 4).

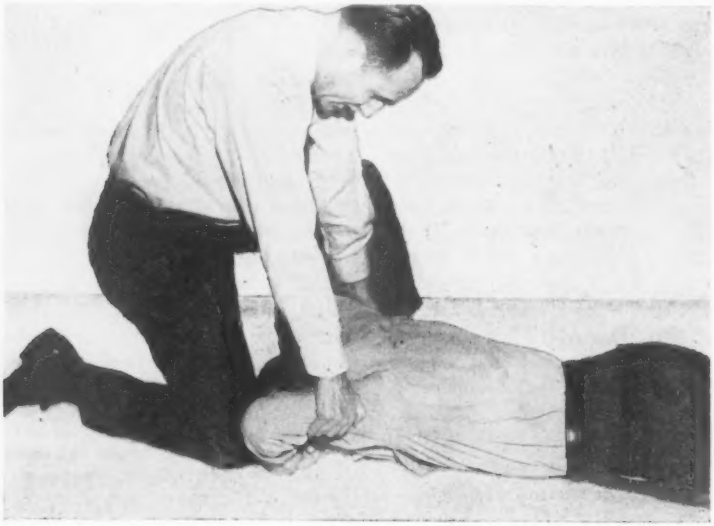

FIGURE 2.

FIGURE 3.

Your elbows should be kept straight. Do not exert sudden or too much pressure, or place your hands high on his back or on his shoulder blades.

Fifth—Release the pressure quickly. This is done by "peeling" your hands from his back without giving any extra push with the release.

Sixth—Repeat these movements rhythmically, lifting arms and pressing

FIGURE 4.

down on the back alternately about 12 times a minute until natural breathing is restored. The operator may time his rhythm at 5 sec. per cycle.

If assistance is at hand, a physician should be sent for; the patient's tight clothing should be loosened at neck, chest, or waist; and the patient should be kept warm. When natural breathing has been restored, treat the patient for shock. The patient should be kept quiet in a prone position until he receives expert medical attention.

Tests for Oxygen Deficiency

There have been several recent references in the sewage works literature to the testing of oxygen deficiency in sewers by the use of Wolf safety lamps. Such lamps may cause an explosion in sewers by the propagation of hydrogen, a constituent of manufactured gas, through the gauze of the lamp. Furthermore, good practice discourages the use of any open flame in the presence of flammable gases, for there is always a possibility that a defective or improperly assembled flame safety lamp may be employed. An oxygen-deficiency indicator, which is an adaptation of the Wolf safety lamp, should be used. With such indicators a sample of the atmosphere under test is brought to the instrument, located in fresh air, by means of a sampling hose and aspirator. The sample is brought into contact with a gauze-protected, bonneted, naphtha-burning flame unit through an intake connection equipped with a flashback arrester capable of preventing the propagation of a hydrogen flame.

Further information on oxygen-deficiency indicators will be found on pages 32 and 45 of Manual of Practice No. 1.

LeRoy W. Van Kleeck,
Chairman, Sub-Committee on Occupational Hazards, Committee on Sewage and Industrial Wastes Practice

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TIPS AND QUIPS

Septic Tank Drainage

One insistent weakness of conventional septic tank drainage fields, especially in areas of clay soil, has been failure of the leaching fields to dispose of the tank effluent. With the usual system percolation is successful enough in sandy soil, but in heavier soils the ditches become clogged with solids. The remedy thus far has been to extend the drainage ditches. On a small lot, however, extension may be inconvenient or impossible.

A solution to the problem is offered in an article in the October, 1951, issue of *Sunset Magazine*. Developed by Fred F. Horne, of Robles Del Rio, Calif., the new system substitutes an 18-in. depth of continuous pumice-soil bed (Figure 1) * for the usual tile-and-trench layout. The pumice (½-in. screened) permanently aerates the leaching field and promotes lateral as well as downward movement of the liquid. The mix runs from 30 per cent pumice in very sandy soil to 60 per cent pumice in heavier soils. The drain tiles are laid in this mixture, just below the surface.

Basically, it's the pumice that makes the system effective. It is absorbent but aerating, and gives more even distribution of the moisture, thus removing the factors making for concentration of root growth that precluded planting of trees and shrubs near old-style drainage fields.

* Reprinted from *Sunset Magazine* by permission.

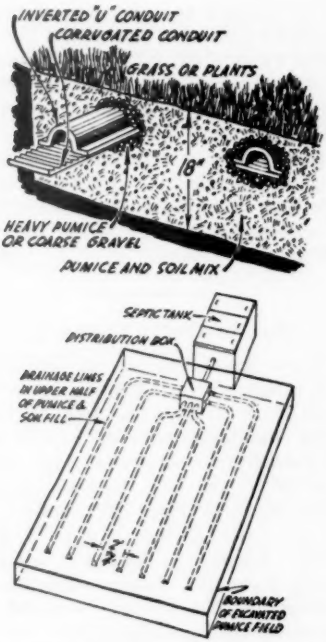


FIGURE 1.—Continuous drain field of pumice-soil mix gives better distribution for septic tank effluent seepage in heavy soils.

Pumice is being used commercially to amend drainage field soils in the West, as it is effective and economical, being available in large quantities as a concrete aggregate. However, there are several inert and lightweight aggregates with considerable pore space that could be used successfully in the East.

Buried Pipe Protection

Protecting buried pipes from corrosion with "Scotch" plastic tapes is the subject of a new 8-page illustrated booklet available upon request from Minnesota Mining and Manufacturing Co., 900 Fauquier St., St. Paul 6, Minn. It describes and illustrates both high-speed machine and hand wrapping techniques for protecting metal pipes, joints, and fittings from the corrosive effect of soil chemicals, oils and fats, stray ground currents, and soil bacteria and fungi.

The booklet cites some of the problems resulting from pipe corrosion, as well as actual case histories of instances where the wrapping methods described have been used. In addition, it gives tables listing the physical and electrical properties of the tapes.

Do's and Don'ts in Hose and Fitting Care

1. Don't expose hose to oil, which rots it. If oil is unavoidable, use special oil-resisting hose. Equip air compressors with filters or separators; lubricate the tool, not the hose.

2. For temperatures greater than 300° F., use asbestos-fabric hose. Heat hardens rubber, causes cracks.

3. Alternate heating and freezing is worse than constant heating.

4. Continuous flexing wears out hose rapidly.

5. Generally, a short length of hose stands more pressure for a longer time than a longer length, a small-diameter hose stands more pressure than a large-diameter one with the same number of plies, and a hose open at one end lasts longer than one coupled to a unit creating pulsating pressure.

6. Don't pass hose where trucks or cars will crush or cut it.

7. Use approved couplings, properly installed, to prevent joint failures due to fabric-rotting or carcass-crushing.

8. Keep high-pressure steam hose under pressure. Shutoff causes condensation and vacuum, which tends to pull the inner tube from the carcass.

9. Use couplings free of burrs and sharp edges.

10. Put rubber cement on coupling stem and inside tube mouth just before assembly.

11. When installing couplings in wire-wound hose, be sure the wire end

is aligned with the hose axis, then held securely by the clamp.

12. Don't cut the hose tube to fit a nipple or coupling stem.

13. Follow the maker's or distributor's recommendations on selecting hose. Don't use wire-wound or double-extra-heavy, etc., just because you want to increase wear life. You may shorten it, and at increased cost.

14. Support hose coils properly when stored, or lay them down on a wooden grill permitting air circulation. Store in a cool, dry, well-ventilated room.

15. Prevent kinking; dragging across gravel, broken stone, or around sharp corners; and exposure to dropped or hot objects, acid fumes, and oil in any form.

New Lube Oil Extends Engine Service Life

A new lubricating oil now being marketed by the Shell Oil Company is said to have been designed to solve certain problems of engine wear and fouling caused by operating conditions that are aggravated by low loads, high-sulfur fuels, and low-temperature operations. Additional advantages are claimed to be most pronounced in certain diesel engines designed for increased power on low-grade fuels.

Known as Shell Rimula Oil, the lubricant is said to extend serviceable engine life by elimination of ring sticking and piston lacquer, and reduction of cylinder, ring, piston, and bearing wear, with subsequent reduction of oil consumption.

Sanitizer for Personal Safety Equipment

Personal safety equipment (masks, clothing, respirators, rubber gloves, etc.) can be cleaned and sanitized thoroughly, quickly, and with a minimum of bother by use of a dry powder "Cleaner-Sanitizer" packed in 1-oz. envelopes. Each envelope contains the correct amount to make the prescribed concentration when added to 1 gal. of warm water (120° F.) The equipment is simply immersed, softly brushed, rinsed in warm water, and allowed to air-dry.

Detailed information on the product is given in Bulletin No. CM–18, available from Mine Safety Appliances Co., Braddock, Thomas and Meade Streets, Pittsburgh 8, Pa.

How to Dodge Lightning

About nine-tenths of the fatalities and injuries from lightning occur in rural districts rather than in areas with many buildings or large buildings. This conclusion is drawn from data supplied by the U. S. Census Bureau, according to the May, 1952, number of *Safety News Letter*, issued by the Public Employee Section, National Safety Council. People engaged in outdoor pursuits are most exposed to the hazards of lightning.

Although lightning usually strikes an isolated object on high ground, such as lone trees, buildings, and the like, it does not always strike the highest object in the area.

What To Do

Upon the approach of an electrical storm, such of the following precautions as fit the situation at hand should be taken:

1. Discontinue operations that are out in the open.

2. Stop loading, unloading, or handling of flammable liquids or products in open containers in exposed places.

3. Discontinue the gaging of tanks containing flammable liquids.

4. Have employees leave the tops of tanks, buildings, and the like.

5. Discontinue work on fences, telephone lines, power lines, structural steel erection, pipelines, or other metal materials that might conduct a lightning charge.

6. Keep away from drain pipes on the outside of buildings, and steel towers, cranes, and other metal objects onto which a charge might be induced.

7. Stay away from the crests of hills, or other places where protection is not afforded by tall objects. Avoid isolated trees, small exposed sheds or outbuildings, wire fences, and telephone or power poles.

8. Seek shelter in large buildings or buildings with protection against lightning, or in a grove of trees, a cave, a ravine, or a gully.

All of which can be summed up by simply saying:

Know where lightning is liable to strike, then stay away from there—unless you're quicker than greased lightning!

ANNOUNCEMENT OF RADIOLOGICAL HEALTH TRAINING PROGRAM

The Public Health Service is sponsoring a radiological health program as a series of short training courses which are conducted at the Environmetal Health Center in Cincinnati. These courses are free of tuition, and are offered to qualified applicants. The training is designed for professional people who are concerned with radiological health problems in their respective fields. Candidates should have a degree in medicine, engineering, or science (physical or biological).

The Radiological Health Training consists of three units or courses of two weeks each, namely: Basic, Intermediate, and Advanced. Completion of the Basic Course (or equivalent training) is a prerequisite for the Intermediate Course, and completion of the Intermediate Course (or equivalent training) is a prerequisite for the Advanced Course. All three courses may be taken consecutively as a six-week unit, or the Basic and Intermediate Courses may be taken consecutively as a four-week unit, or any course may be taken individually, providing entrance requirements are fulfilled.

The Basic Course provides training in the theory of radiation and radiation detecting instruments, the calibration and use of these instruments, the detrimental effects of radiation, the means of shielding and protection against radiation, and permissible radiation exposure levels.

Basic Courses are scheduled for October 6–17, 1952; January 19–30, 1953; and April 20–May 1, 1953.

The Intermediate Course provides training at a more advanced level. This course is designed for those who have had the equivalent of the Basic Course and covers area and personnel monitoring, the radioassay of water, food, sewage and biological samples, and the operation, maintenance, and repair of radiation detection instruments.

Intermediate Courses are scheduled for October 20–31, 1952; February 2–13, 1953; and May 4–15, 1953.

An Advanced Course is being presented for professional personnel who are concerned in particular with *occupational* radiological health problems. In this course, the present and probable future applications of radiation in industry and certain institutions are discussed. The radiation phenomena, associated health hazards, recommended protection principles, and applied instrumentation are also covered. The Advanced Course is offered to persons who satisfactorily have completed the Intermediate Course, or an equivalent training program.

The Advanced Course is scheduled for February 16–27, 1953.

Further details and information can be obtained from the Radiological Health Training Section, Public Health Service, Environmental Health Center, 1014 Broadway, Cincinnati 2, Ohio.

Federation Affairs

SILVER ANNIVERSARY MEETING TAKING SHAPE

A quarter of a century of progress in sewage and industrial wastes treatment, as well as a quarter of a century of steady and solid growth in membership and accomplishment, will have a fitting climax in the Twenty-Fifth Annual Meeting of the Federation of Sewage and Industrial Wastes Associations, to be held at the Hotel Statler, New York, N. Y., October 6–9, 1952. Preparations for the gala Silver Anniversary Meeting are already in the "final details" stage, with many metropolitan area members of the host New York Sewage and Industrial Wastes Association at work on arrangements.

Local Committees

The Local Arrangements Committee, under the chairmanship of George E. Symons, consultant and technical editor, Larchmont, N. Y., is composed of the chairmen of the following subcommittees:

Registration Committee: C. R. Velzy, chairman; R. D. Nickerson, W. T. Ingram, J. J. Baffa, and H. P. Orland.

Finance Committee: T. T. Quigley, chairman; J. C. Brigham, R. C. Sweeney, and J. Porter.

Inspection Trip Committee: W. A. O'Leary, chairman; N. I. Kass, S. W. Steffensen, and W. L. Sylvester.

Entertainment Committee: J. J. Horgan, chairman; J. G. Stewart, C. K. Perkins, J. D. Bowe, and A. O. Hopper.

Ladies Entertainment Committee: Mrs. Karl Mann, chairman; Mrs. G. E. Symons, Mrs. R. Hazen, Mrs. C. R. Velzy, Mrs. H. A. Faber, Mrs. S. G. Hess, Mrs. M. M. Hirsh, Mrs. J. J. Horgan, Mrs. H. S. Sutton, Mrs. E. M. Jones, Mrs. A. L. Morris, Mrs. R. D. Nickerson, Mrs. W. A. O'Leary, Mrs. R. F. Orth, and Mrs. T. T. Quigley.

Local Host Committee: S. G. Hess, chairman; T. M. Riddick, A. J. Fischer, J. M. Brown, Jr., F. C. Roe, D. M. Ditmars, and A. N. Diachishin.

Hotel Arrangements Committee: H. A. Faber, chairman.

Early Registration

To help early arrivals at the convention avoid the usual last-minute registration rush on Monday morning, it has been arranged to have the registration desk open on the hotel mezzanine from 2 PM to 5 PM on Sunday, October 5. Pre-registration is urged for all those in town during this period.

Announcement to Membership

All Active and Corporate Members of the constituent associations of the Federation have been sent a formal announcement of the convention. Contained therein are further details as to technical and social program, registration, entertainment events, and manufacturer exhibits, as well as a reservation card for hotel accommodations. It should be noted that although the meeting will officially close at noon on October 9, the inspection trips are scheduled for the afternoon of that day. Reservation of hotel accommodations should be placed directly with the Hotel Statler in New York City and should be made early.

Reviews and Abstracts*

Determination of Organic Nitrogen in Sewage, Sewage Effluents, and River Water. By S. H. Jenkins. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 147 (1950).

The author describes the detailed method of determining organic nitrogen, then recommends that addition of ammoniacal, oxidized (nitrite plus nitrate), and organic components will determine total nitrogen.

K. W. Cosens

Determination of Ammoniacal Nitrogen in Sewage, Sewage Effluents, and River Water. By S. H. Jenkins. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 144 (1950).

The author suggests a distillation titration method using N/140 H_2SO_4 and N/140 NaOH, which he considers quicker, more accurate and more flexible, except for relatively pure river waters, than present "standard methods" using Nesslerization.

K. W. Cosens

Amperometric Titration of Copper and Ferric Iron with Cupferron. By I. M. Kolthoff and A. Liberti. *Analyst*, 74, 635 (1949).

Conditions for using cupferron reagent for the quantitative determination of cupric copper, ferric iron, and mixtures of the two have been studied. Cupferron solutions prepared in conductivity water are stable for at least 14 days if kept in dark bottles. Procedures are described for amperometric titration of the metals, in suitable buffer solutions, with cupferron. The accuracy is 1 per cent in 0.001M solutions. The accuracy of the iron titration is not affected by the presence of large amounts of aluminum, zinc, trivalent chromium, manganese, and cadmium. Copper can be titrated in the presence of a large excess of iron in a citrate buffer that is 0.5M with respect to potassium hydrogen fluoride. When the ratio of iron to copper is equal to or greater than 1, iron can be titrated in a medium of 0.1N hydrochloric acid. The end point is reached before copper is precipitated.

H. P. Orland

Modified Stockport Bio-Aeration Units (Comparative Results) By F. C. Holt. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 100 (1950).

Operating results are tabulated for three variations of the activated sludge method of sewage treatment. Average values of 4-hr. oxygen absorbed at about 15 p.p.m., 5-day B.O.D. equal to about 17 p.p.m., and suspended solids at about 16 p.p.m. were given as a measure of the strength of the plant effluent. Plant influent is described as being strong and composed of 20 per cent trade wastes.

K. W. Cosens

Development of Waste and Sewage Problems in the Industrial Area of Twenthe (Holland). By J. J. Hopmons and G. S. Bos. *Proc. Royal Inst. Eng.* (Netherlands), Nos. 33–35, 541–77 (1951).

In 1897 a government commission reported that the pollution in the Twenthe area was primarily caused by textile wastes. After considerable litigation, the city of Enschede constructed a sewage treatment plant in 1937, followed by other municipalities in the area. The status of treatment in 1950 was as follows:

* Please send to Federation headquarters all periodicals, bulletins, special reports, etc., which might be suitable for abstracting in This Journal. Publications of public health departments, stream pollution control agencies, research organizations, and educational institutions are particularly desired. Address such material: Federation of Sewage and Industrial Wastes Assns., 325 Illinois Bldg., Champaign, Ill.

Plants	No.	Pop.	Ind. Pop. Equiv.	Total	Flow (m.g.d.)
Installed	5	103,000	240,000	343,000	10.2
Projects ready	7	84,500	86,500	171,000	5.3
In preparation	8	74 000	92,000	166,000	7.4
Total	20	261,500	418,500	680,000	22.9

The smaller plants consist of settling and rotary trickling filters. The largest plant (Enschede), with a flow of 5.8 m.g.d. and 580,000 gal. per hr. maximum flow, capable of treating completely three times the dry weather flow, consists of automatic screen, grit chamber, pre-aeration (Kessener system), 12 tricking filters, humus tanks with flocculation, heated three-stage digesters, and appurtenances. The gas produced is used for power supply for two gas engines of 290 hp. each; exhaust gases are used for heating the digesters and excess gas is sold to the municipal gas works. Digested sludge is dried on open beds. The plant treats a strong sewage, containing 1,500 p.p.m. total solids, 500 p.p.m. settleable solids, and 245 p.p.m. chlorides. The B.O.D. of the raw sewage is 625 p.p.m.; of the settled sewage, 435 p.p.m. The digesters are of a new type, wherein the raw sludge is pumped to a tower connected with piping to the stage digesters. Normally, the sludge is discharged from the tower to the preheater, located in the center of the first tank. The preheater is equipped with rotating bundles of pipes for hot-water heating (digestion temperature 95° F.). The sludge flows continuously over the weir into the second-stage digester, covered with a movable (rotating) cover, to which the heating coils are attached. The third-stage tank has a "screw" gas holder to produce higher pressure for the gas scrubbers. Special foundations were necessary to prevent floating of empty tanks during construction. W. RUDOLFS

Report of Water Pollution Study of Miami River — 1949. Ohio Department of Health (Jan., 1951).

The main objective of the survey was to obtain information upon which regulatory pollution measures could be based. Regulatory measures should restore and preserve the streams of the basin for uses consistent with public health and welfare requirements.

The amounts of deoxygenating, chemical, and metallic wastes were determined by analyses at many points in the river basin. By projecting the demands of these wastes into minimum stream flow conditions at appropriate points on the river, it was possible to make recommendations regarding degree of treatment required to maintain a minimum dissolved oxygen content of 4.0 p.p.m.

An analysis of the Miami River basin is given in terms of its geography, geology, physiography, ground-water storage, flood control and other water developments, population, and industrial development.

A discussion is given comparing the classical method of Streeter and Phelps with the newer method of Le Bosquet for analyzing the self-purification capacity of a river.

Each of 184 sampling stations established on the river at strategic points was sampled from three to nine times. Sampling and analysis methods followed "Standard Methods."

Recommended critical or design minimum stream flows were given for the main stream and its tributaries at the important sources of pollution. These design minimum flows were based upon the smallest expected 7-day average flow which would occur with a frequency of about once in 5 years. Basic hydrological data were furnished by the U. S. Geological Survey.

Of particular interest are two sets of curves showing minimum dissolved oxygen in the Miami River in 1933–36 and again in 1949. During this time considerable improvements were made in the Dayton sewage treatment plant and the results clearly show on the D.O. curves. The river immediately below Dayton was in much better condition in 1949 than it was in 1933–36. However, there were other places where new treatment facilities had not been installed and river conditions were still unsatisfactory.

Results of analyses for metallic and

chemical pollution in the various metropolitan areas along the river are given. Special emphasis was placed on phenol, cyanide, iron, copper, lead, zinc, chromium, pH, and the discharge from an oil refinery. It was clearly brought out that there was at present no good method of estimating population equivalent of such wastes as phenols, cyanides, and others which have no B.O.D. counterpart.

KENNETH W. COSENS

Pollution of Streams: Some Notes on Recent Research. BY B. A. SOUTHGATE. Inst. of Civil Eng. (British), Pub. Health Eng. Div. (Oct., 1950).

One of the newer developments in treating sewage is the use of mechanical flocculation without the addition of chemicals. Studies with laboratory apparatus have shown that 61 per cent of suspended matter was removed by mechanical flocculation followed by sedimentation and that the effluent contained from 11 to 22 p.p.m. of suspended solids.

A report was also made on the results obtained when humus tank effluent was filtered through gravity filters quite similar to rapid sand water treatment filters. On filters at Coventry, with sand passing No. 8 mesh and retained on a No. 16 mesh screen, it was found that, at 150 gal. (Imp.) per sq. ft. per hr., suspended solids were reduced 85 per cent (from 39 to 6 p.p.m.) and that B.O.D. was reduced 70 per cent (from 36 to 11 p.p.m.). Better results were obtained at 100 and poorer results at 290 gal. per sq. ft. per hr. loading. It may be noted that the area required by rapid sand filters operating at 150 gal. per sq. ft. per hr. would be only 0.6 per cent of the area required of trickling filters 6 ft. deep operated at 100 gal. (Imp.) per cu. yd. per day.

The reduction of the bacterial load of sewage effluents is discussed and two methods are suggested for the reduction of bacteria—chlorination and sand filtration. Results of the two methods indicate that chlorination is much more effective in removing the coliform group of bacteria than is the filtration method.

In connection with the discussion of toxic substances found in sewage plant effluents, it is pointed out that ammonia is toxic to fish and that the effect is even more important as the higher pH values are encountered. Experiments conducted in England in water containing ammonium chloride, equivalent to 41 p.p.m. of nitrogen, showed it was lethal to trout in 30 hr. at pH 7.7. A. E. Cole, in the Wisconsin area, reports that 6 to 8 p.p.m. of nitrogen is lethal. Apparently, this is a problem that needs considerable more study. The margin between the quantity of DDT needed to control filter flies and that which will be toxic to fish is very small. Therefore, these insecticides must be used with caution.

Several interesting aspects of the effects of discharging unpolluted cooling waters to streams are discussed. The sewage fungi grow more rapidly at elevated water temperatures, their presence having several undesirable effects. When the stream temperature is elevated, B.O.D. increases rapidly; for instance, 5-day B.O.D. at 30° C. is 28 per cent greater that at 20° C. In polluted rivers the warmer waters cannot retain as much oxygen as can the cold waters. Therefore, cooling water affects the D.O. capacity of a stream, regardless of organic load. The author also points out that the toxicity of cyanide approximately doubles as temperature rises from 8° to 18° C.

KENNETH W. COSENS

Waste Problems in the Mineral Industries. BY JAMES BOYD. *Chem. Eng. News*, **29**, 467 (1951).

This paper was presented as an address before the Air and Stream Pollution Section of the Twelfth International Congress of Pure and Applied Chemistry. The article summarizes the present situation of acid mine drainage, oil field brines, and silt deposits in water channels. Air pollution is also discussed. R. S. INGOLS

Sewage Purification by Rock Filters—Performance of Standard-Rate Trickling Filters. BY J. H. SORRELS AND P. JI A. ZELLER. *Bull.*, Agric. and Mech. Coll. of Texas, **6**, 1 (Nov. 1, 1950).

Trickling filters receiving loads of 650 lb. B.O.D. per acre-foot per day produce effluents of similar quality to those produced under 450-lb. loading. Loads up to 1,000 lb. B.O.D. per acre-foot per day may be handled with moderate additional dilution in the receiving stream, but efficiency decreases with loadings of more than 1,000 lb. Settled sewage was applied to four experimental rock filters

at varying B.O.D. loads; rate of flow was different, but was held constant for each filter. Criteria for determining filter performance were removal of applied B.O.D., removal of applied suspended solids, and formation of nitrates. Composite curves are given for use in filter design. H. P. ORLAND

Determination of Oil in Refinery Waste Waters, A Benzene-Extraction Method. By A. F. S. MUSANTE. *Anal. Chem.*, **23**, 1374 (1951).

The method described uses large samples for wet extraction and separates the solvent from the extracted oil by distillation instead of using an open evaporating dish. Intimate mixing of the solvent with the sample is achieved by a motor-driven impeller mounted on a cork, which fits into a regular 1-gal. jug. The sample is 3 l. and the shaft of the motor extends about 2 in. into the water layer. At least two extractions with the solvent are made and then the entire sample is placed in a separatory funnel. The aqueous layer is discarded and the benzene layer is placed in a standard taper flask with an air condenser, which leads into the open end of a water condenser. After removing most of the solvent, the residue is transferred into a tared miniature still assembly heated with a closely wrapped coil. A thermocouple in a thermometer well is placed in the flask to check the temperature rise when the solvent has been completely evaporated. The thermocouple is particularly important where the sample contains low-boiling oils. The method gains its particular advantage in the separation of the low-boiling oils from the solvent where it is shown that kerosene is completely lost by trying to separate the solvent from kerosene with an open dish evaporation of the solvent from the residue. The method shows good sensitivity and accuracy at known oil values in the range of 10 to 25 p.p.m.
R. S. INGOLS

Magnesia-Base Sulfite Pulping. By W. Q. HULL, R. E. BAKER, AND C. E. ROGERS. *Ind. Eng. Chem.*, **43**, 2424 (1951).

In a recent survey of the pulp and paper mill industries' waste disposal problem, a leading authority stated that the problem would be solved in 10 years. This report of a 275-ton per day production of paper by magnesia-base sulfite pulping is one of the reasons for optimism about the reduction in stream pollution by the pulp and paper industry.

Calcium sulfite pulping wastes are not subject to concentration and re-use for two major reasons: the scaling of the evaporator equipment and the formation of calcium sulfate in the combustion of the concentrated process waters. The magnesia sulfite pulping wastes answer these two problems, because the magnesium salts are so much more soluble than calcium and in the combustion of the concentrated process waters there is a chemical disintegration to magnesium oxide, which can be recovered from the ash and stack, as well as sulfur dioxide, which can be reclaimed. The flow sheet of the magnesia sulfite pulping mill looks very much like that of a kraft paper mill, except that there is a large sulfur dioxide recovery step that is lacking in the kraft mill.

No direct figures on the nature of the plant wastes are given in the article, but there is a consumption of 85 lb. of sulfur per ton of paper at the present time, whereas calcium-base pulping consumes 270 lb. of sulfur per ton of paper. It is expected that as the capacity of the recovery mill is increased to meet the actual paper production use of make-up sulfur will be reduced still further. The weight of magnesia per ton of paper is only 50 lb., as against 350 lb. of lime rock per ton. Thus, at least two-thirds of the waste from a calcium sulfite mill is being used for heat at the paper mill using magnesia base pulping. R. S. INGOLS

Portable Meter for Determining Dissolved Oxygen. By G. A. MARSH. *Anal. Chem.*, **23**, 1427 (1951).

In the disposal of oil field brines, it is desirable to reduce their corrosiveness. This is done by aerating to remove carbon dioxide and then de-aerating to remove dissolved oxygen. It is very important to have some means that can be used in the field for checking dissolved oxygen which is not influenced by the dissolved salts of the brine. A simple portable polarographic instrument has been developed which uses a rotating platinum electrode against a calomel reference cell. Dissolved oxygen values can be obtained in

natural brines with an accuracy of ± 5 per cent. The instrument is operated by batteries and has some electronic magnification for stability.

It is stated that the instrument does not operate with a straight-line calibration in fresh water, but does operate at various salt concentrations above 1 per cent with no change in calibration. The instrument has been successfully used in controlling dissolved oxygen at brine handling plants. R. S. INGOLS

Industrial Wastes—Ozone Provides an Economical Means for Oxidizing Phenolic Compounds in Coke Oven Wastes. BY HAROLD R. MURDOCK. *Ind. Eng. Chem.*, **43**, 125A (NOV., 1951).

Because ozone does not attack ammonia and some of the other constituents of ammonia still wastes, it can compete economically with chlorine in destruction of phenols in these wastes. Where 5,000 to 6,000 p.p.m. of chlorine are required, only 500 to 600 p.p.m. of ozone are required to destroy the phenols. Further, ozone is so unstable that no residual effect will carry over in the treatment plant effluent, whereas some of the chlorine compounds are very persistent and control of the effluent must be considered. At the moment, chlorine is in short supply, whereas ozone can be generated from air or oxygen gas and electrical power. Even with the cost of equipment for generating ozone considered, the cost of ozone over a period should be able to show a favorable cost balance. R. S. INGOLS

Summary Report on Water Pollution: California Drainage Basins. U. S. Public Health Service Publ. No. 86 (1951).

This fourth in a series of summary reports on major drainage basins of the United States points out that sewage and waste treatment projects are needed at 220 municipal and industrial locations in the California Basin to bring water pollution under proper control in an area including nearly all of California and part of southern Oregon, from the Smith and Klamath Rivers on the north to the Mexican border on the south.

A total of 529 municipalities, institutions, and sanitary districts discharge sewage, including industrial wastes discharged through public sewer systems, with a population equivalent of 8,500,000. There are also 443 known separate industrial waste outlets in the basin. Of these, 83 produce organic wastes with a population equivalent of 2,800,000; 297 produce organic wastes of undetermined population equivalent; and 63 produce inorganic wastes or wastes of undetermined type.

On the basis of available data, 220 sewage and waste treatment projects are needed in the basin, including 204 for municipalities, with population of 2,755,000, and 16 for industries. Many of the municipal plants treat industrial as well as domestic wastes. Indicated needs for municipalities include 84 new treatment plants, enlargements or additions to 62, and replacement of 58 plants. Because of the scanty information available on industrial wastes, the known treatment plant needs in industry, indicated in the report as 9 new plants, 6 enlargements or additions, and 1 replacement, are undoubtedly only a small part of the total needs.

No projects are needed at 232 municipal and 38 industrial locations, but project needs are undetermined for 13 municipalities and 389 industries. Sewage and waste treatment facilities are provided by 438 municipalities and 199 industries.

No estimate of costs for the indicated needs is made because California law prohibits specification by the authorities of type and degree of treatment, which are important factors in arriving at costs. Limited information on the industrial waste treatment needs also would make any cost estimate misleading. However, a range of cost figures (from $10 to $60 per capita) is suggested for projection of the 2,750,000 population to total cost of the improvements. This is exclusive of the industrial waste treatment costs.

H. P. ORLAND

Effluents From Fermentation Industries: Part I. A Review of Methods of Disposal and Utilization. BY G. J. STANDER. *Pub. Health* (So. Africa), **14**, 8, 243 (1950).

The National Chemical Research Laboratory (So. Africa) conducts investigations to enable industries to overcome disposal problems. The results of an investigation on disposal of distillery wastes are presented in five parts.

A review of literature shows that some distillery residues can be used to produce vitamins, animal feeds, or tartaric acid, but that such procedures are not always economical. Results of studies and treatment methods, mainly those used in the United States, are reviewed, resulting in the conclusion that, although there is a lack of usable and constructive data published, treatment by biological methods and in particular by the anaerobic fermentation process holds out definite promise.

Part II. The Significance of the Solid Phase and of Volatile Acid Development in the Anaerobic Method of Treatment. By G. J. Stander and C. G. Hide. *Pub. Health* (So. Africa), **15**, 2, 38 and **15**, 3, 71 (1951).

A review of research on digestion shows that a solid phase is beneficial and that volatile acid production should be limited. Tests covering these factors were made in flasks set in thermostatic baths at 55° C.

Distillation residues from the production of ethyl alcohol, from both sugar cane molasses (slop liquor) and from grape juice (still liquor) were used. Tables covering each type of waste and test show mineral composition, general analyses, composition of supernatant after 6 and 206 days' feed, volatile acids and gas production for each rate of feed, and reduction of organic carbon after 45 days at the highest rate (30-day digestion period).

In the first series, four flasks were charged, each with 1,500 ml. of filtered sewage digester supernatant, and 100 g. of powdered asbestos, bentonite, coal and kieselguhr, respectively, and the fifth with only digester supernatant. When maintained 4 to 5 days as a closed system, 500 ml. of filtered inoculum prepared from molasses slop was added. Feeding started 48 hr. later at 1 ml. of slop liquor per day and was gradually increased to 50 ml. Settled supernatant was analyzed.

On straight slop liquor, the absence of a solid phase did not retard gas production, but a solid phase was produced in the blank digester. Comparison of results from each digester showed few significant differences. On the basis of 4-hr. oxygen consumed (no B.O.D.'s were run), tests with asbestos showed the greatest reduction (67.4 per cent). Reduction for others, including the blank, varied from 59.0 to 62.1 per cent. In all cases the rise in volatile acids was rapid, varying from 600 to 1,145 p.p.m. at a feed of 5 ml. per day.

The sulfate concentration of the slop was reduced from 6,750 to 500 p.p.m. and tests were run using blank and asbestos-treated digesters. Results were much better than before and the solid phase showed no advantage. The proportion of organic carbon in the feed converted to gas averaged 67.6 per cent for treated wastes and 54.7 per cent for the untreated. The rate of feed was increased to the maximum more rapidly for the treated, with little increase in volatile acids (340 p.p.m. maximum). These series proved that sulfates in the slop had a retarding effect on digestion.

It was concluded that the organic concentration of wine still liquor falls within the range of sewage sludge, and the sulfate content is only 400 to 500 p.p.m. Tests were conducted as before and repeated in three digesters. Conversion of organic carbon to gas ranged from 76.6 to 84.8 per cent, except in the blanks, where it was 69.7 and 70.1 per cent. Only the blank digesters showed signs of deterioration before the end of the test, volatile acids reaching 2,180 p.p.m. at a feed of 30 ml. per day. For the solids phase tests, only once, at the 50-ml. rate, was the volatile acids content high (1,430 p.p.m.).

It was concluded that the rise in volatile acids was due to an unbalance in the complex bacterial activity, brought about by large quantities of sulfates, rather than to high rates of feed. A discussion is devoted to factors affecting bacterial balance during digestion.

Part III. The Influence of High Concentrations of Sulphates in the Anaerobic Digestion Method of Treatment at Thermophilic (55° C.) and Mesophilic (33° C.) Temperatures. By G. J. Stander and J. F. Elsworth. *Pub. Health* (So. Africa), **15**, 9, 198 and **15**, 12, 267 (1951).

Studies were repeated on molasses slop high in sulfates, to determine the influence of sulfates, the differences between mesophilic and thermophilic digestion of liquors containing high and low concentrations of sulfates, and the degree of conversion of sulfates to H_2S.

Digestion flasks containing asbestos were prepared and operated at 55° C. as

described in Part II. Inoculum using neutralized slop was likewise prepared as described. De-sulfated slop and normal slop with the pH adjusted to that of the de-sulfated was first used. Results showed that it was possible to increase the feed of de-sulfated slop to 50 ml. per day without much rise in volatile acids, whereas on the normal slop it could not be increased above 20 ml. per day. A difference of 15.5 per cent in the amount of organic carbon converted to gas clearly showed the inhibitory influence of the sulfates.

For the next series de-ionized molasses slop was divided into three portions, which were then re-mineralized alike, and to which were also added: to No. 1, yeast extract; to No. 3, sufficient ammonium sulfate to give a sulfate content over 5,000 p.p.m.; and to No. 2, ammonium chloride to give the same ammonium nitrogen content as in No. 3. Two flasks of each were prepared, one digested at 55° C. and the other at 33° C.

At 55° there was a marked difference in the percentage of organic carbon converted to gas (37.4 per cent in sample No. 3, compared to 57.8 per cent in each of the others). Volatile acids increased rapidly in No. 3, rising to 4,100 p.p.m. in 48 days, with a maximum loading of 50 ml. per day; the other digesters were operated longer at a loading of 80 ml. per day with hardly any rise in volatile acids. At 33° the inhibiting effect of the sulfate on gas production was demonstrated, but its presence did not seem to contribute greatly to the volatile acid increase. It was also found that when the rise in volatile acid production was accompanied by a drop in pH, gas production decreased rapidly, but a rise in volatile acid production with no change in pH did not affect gas production.

The "sulfate ion was reduced almost completely by thermophilic and mesophilic anaerobic fermentation." The percentage of sulfate collected as H_2S was highest at 55°, but the average percentage of H_2S in the gas was the same at both temperatures, being approximately 0.42 per cent for sample No. 3.

Part IV. New Method for Increasing and Maintaining Efficiency in the Anaerobic Digestion of Fermentation Effluents. By G. J. Stander. *Pub. Health* (So. Africa), **14**, 9, 263 (1950).

The active bacterial population in a sewage sludge digester is developed from raw sludge by establishing favorable environmental conditions and the addition of raw sludge is, in effect, a re-inoculation. In feeding waste liquors such as molasses slop, containing only specific organisms of manufacture, to an active digester, no fresh supply of bacteria for anaerobic digestion is added. A method of operation was studied involving the addition of an inoculum to remedy the bacterial unbalance indicated by volatile acid development.

Tests were made using still liquor, and apparatus and procedure the same as that used for previous studies. Inoculum was prepared by feeding small amounts daily of still liquor and raw sewage sludge to a separate digester started from supernatant sewage digester liquor, powdered asbestos, and actively digesting sludge.

Two batches were digested at 33° C. and one at 55° C. Feeding continued to a digestion period of 7.5 days, and when volatile acid production started increasing, 200 to 300 ml. of supernatant was withdrawn and replaced with the inoculum. In all instances the volatile acid concentration was effectively controlled. At 55° an average of 88.7 per cent organic carbon was converted to gas and there was a reduction of 92.5 per cent oxygen consumed (4 hr.) At 33° the averages were approximately 82 and 95.5 percent, respectively.

Tests also were made using wastes from the manufacture of bakers yeast, from cane sugar, molasses, and malt. The high sulfate content (1,380 to 2,700 p.p.m.) caused difficulties in preparing the inoculum, which were overcome by starting with still liquor.

Experiments at 33° were discontinued at a digestion period of 3.75 days with the digester still in good condition. At 55° the digester failed when a 30-day digestion period was reached, and addition of inoculum could not restore it. At earlier stages, though, the inoculum did help. At 33°, addition of inoculum immediately arrested the rise of volatile acids, the highest measured being 385 p.p.m. At 33°, conversion of organic carbon to gas was 53.7 per cent and reduction in oxygen consumed 73.6 per cent. At 55° these figures were 38.8 per cent and 58.9 per cent, respectively. Complete tables are given.

General consideration of results shows that the ammonia nitrogen concentration of the digester liquors had very little effect on gas production. Although digester liquors were still stronger than domestic sewage, they were made more amenable to aerobic treatment.

WM. A. HASFUTHER

Part V. Re-Inoculation as an Integral Part of the Anaerobic Digestion Method of Purification of Fermentation Effluents. BY G. J. STANDER AND R. SNYDERS. *Pub. Health* (So. Africa), **16**, 2, 28; **16**, 3, 47; **16**, 4, 89; and **16**, 5, 109 (1952).

Experiments were made using still liquor, yeast waste, and molasses slop, with re-inoculation practiced as an integral part of the digestion and not as a remedial measure only. Inoculum digesters fed with each type of waste and fresh sewage sludge were maintained for the duration of the experiments.

A digester flask for each waste was started from actively digesting sewage sludge and inoculum. After regular feeding started, 150 ml. of inoculum was added every 48 to 96 hr., depending on the daily feed volume. At high feeds (33° C.) gasification was so vigorous that the solid phase failed to settle out and the supernatant liquors removed were centrifuged and the sediment returned to the digester.

Wine still liquor and yeast waste experiments were discontinued when the digestion period reached two days, although results indicated that a higher feed could be used. Molasses slop tests were discontinued at a digestion period of 3.75 days because of the high oxygen consumed value of the effluent, although otherwise there was no sign of deterioration.

The following loadings (lb. per cu. ft.) were calculated on the basis of maximum feed:

	Still Waste	Yeast Waste	Molasses Slop
Oxygen cons.	0.281	0.108	0.21
Tot. solids	0.855	0.475	0.80
Vol. matter	0.73	0.372	0.546
Org. matter	0.462	0.191	0.267

Per cent reductions in the waste liquor before and after digestion at the maximum rate follow:

	Still Waste	Yeast Waste	Molasses Slop
Org. C to gas	85.5	42.8	48.6
Org. Carbon	95.3	81.8	66.5
Sulfate	61.5	94.2	87.4
Oxygen cons. (4 hr.)	96.4	75.3	69.7

At no time did the volatile acid content get too high for good operation, and even with molasses slop at a sulfate concentration of 2,865 p.p.m. only once did the volatile acids exceed 1,000 p.p.m.

Sludge in all digesters increased noticeably. That it carried a considerable proportion of the active bacterial population was evidenced by a steep rise in volatile acid production and drop in gas production when about 25 per cent of the sludge was removed in three days.

In the case of both still liquor and yeast waste the organic nitrogen decreased considerably, but no accumulation of ammonia was observed.

These papers also include discussions on nitrogen changes, gas content, solids, and pH, and are replete with full tables of results worthy of careful study.

WM. A. HASFURTHER

Lake Erie Pollution Survey—Interim Report. Dept. of Natural Resources, State of Ohio (Apr. 1951).

The primary purposes of the survey were the determination of chemical, physical, and bacterial quality of Lake Erie waters and of the waters of the principal rivers discharging into Lake Erie, from Ohio basins, and to recommend methods of elimination or reduction of pollutants. Work of the survey was conducted by the Division of Water, Ohio Department of Natural Resources; Division of Engineering, Ohio Department of Health; and the Franz Theodore Stone Laboratory. Studies extended from June through September 1950.

Sampling and testing for coliforms at the waterworks intakes along the Ohio side of Lake Erie showed the water in all cases to be of satisfactory quality for domestic supply, provided adequate treatment followed. The coliform count, in indicated numbers per 100 ml., was well under 500 on a seasonal average basis. The maximum allowable is 5,000 per 100 ml.

The results of sampling at bathing beaches were not nearly so satisfactory. At 11 out of 25 bathing beaches the seasonal average of coliform was above 1,000 per 100 ml., the maximum allowable. The greatest pollution occurred in the metropolitan Cleveland area and extended 25 or 30 miles either way from the area. The seasonal average at 7 beaches within the immediate Cleveland area was approximately 30,000 coliform per 100 ml., which is alarming.

Dissolved oxygen and B.O.D. examinations of tributary waters were made and it was found that the Maumee and the Cuyahoga Rivers had B.O.D. in excess of dissolved oxygen content, indicating gross pollution.

Chemical and physical properties of tributary and lake water were studied and included the following analyses: discharge, water temperature, suspended sediment, dissolved solids, oxygen consumed, dissolved oxygen, phenol, cyanide, color, pH, specific conductance, silica, iron, copper, chromium, calcium, magnesium, sodium, potassium, carbonate, bicarbonate, sulfate, fluoride, nitrate, and hardness. Each major drainage basin was studied in light of the above analyses, and causes and character of pollutants pointed out.

Suspended sediment stations were established on four rivers discharging into Lake Erie and provisional results of the suspended silt load were reported.

KENNETH W. COSENS

New Copper-Specific Group. By J. HOSTE. *Anal. Chim. Acta*, **4**, 23 (1950).

In searching for a specific reagent for copper, a number of compounds were prepared and tested. 2-2′ diquinolyl was found to form a copper complex, which can be extracted from aqueous solution by amyl alcohol. The reaction is specific and sensitive and can be used for the determination of copper, even in the presence of a large excess of other ions, including iron and nickel. A method of using 2-2′ diquinolyl for the detection and colorimetric determination of copper is described. H. P. ORLAND

Spectrophotometric Determination of Copper as Diethyldithiocarbamate. By T. C. J. OVENSTON AND C. A. PARKER. *Anal. Chim. Acta*, **4**, 135 (1950).

Dialkyldithiocarbamates, which form colored complexes with many common metals, can be used in the determination of copper. Ammonium citrate is added to the solution containing the precipitate of copper and the complex is then extracted with carbon tetrachloride. The absorption spectrum shows a maximum at 4,360Å. and is unaffected by other metallic absorptions at this wavelength. Because the absorption of the colored complex is destroyed by exposure, light should be excluded as far as possible during the determination. Corrections must be made for the presence of bismuth, cobalt, and nickel. The method may be used for amounts as small as 0.2 g. of copper.

H. P. ORLAND

Some Notes on the Analysis of Sewage and Trade Effluents. By M. LOVETT AND H. FISH. *Jour. and Proc.*, Inst. Sew. Purif., Part 1, p. 5 (1950).

Some of the simpler, shorter, or perhaps less accurate methods of analysis, together with limitations and precautions regarding each method, are listed by the authors or discussers. The following methods are presented:

1. Indicators and pH determinations. Objectives of new indicators are sharper end points, particularly when titrating colored solutions. Indicator papers for pH are discussed.
2. Oxygen absorbed.
3. Ammonia nitrogen.
4. Cyanide.
5. Thiocyanate.
6. Metallic substances, including ferrous and ferric iron, copper (four methods), chromium, and chromate.

K. W. COSENS

Synthetic Detergents and Sewage Processing. By A. LAWRENCE WADDAMS. *Jour. and Proc.*, Inst. Sew. Purif., Part 1, p. 32 (1950).

Detergent processes in general are explained, with the differences between soaps and synthetic detergents being pointed out. Discussion of the various types of detergents includes the economic aspects involved in their use. It is pointed out that diminishing oil and fat production has added to the demand for detergents in both the domestic and industrial markets. Increasing demands for detergents also are based on the merits and

applications of the synthetic detergents themselves.

Negligible effects on primary sedimentation are shown at concentrations below 40 p.p.m. sodium secondary alkyl sulfate. Use of detergents in concentrations as high as 100 p.p.m. sodium secondary alkyl sulfate produces no bactericidal effect on *B. coli* and *B. proteus*, but some lethal effect has been produced on *Staph. aureus* at half this concentration.

Preliminary reports indicate that high concentration of detergents does not affect anaerobic digestion. Little effect on settlement of humus sludge is expected, as most of the detergents precipitate in the primary sludge and the remainder largely decomposes on trickling filters.

Grease recovery effects are not reported.

No mention is made of the activated sludge process or any form of aeration and the effects of detergents. (F. I. Norris, Cincinnati, Ohio, reports that 6 p.p.m. of one detergent causes foaming of aeration tanks.)

The author suggests chemical or mechanical flocculation as an aid in treatment in case of exceptional concentrations of detergents. Discussers pointed out that 40 p.p.m. of sodium secondary alkyl sulfate was enough to cause complete failure of sewage treatment plants depending on acid precipitation. Discussers agreed that 20 p.p.m. of sodium secondary alkyl sulfate put about 50 per cent greater B.O.D. load on secondary treatment processes. It also was agreed that trickling filters removed almost all detergents coming to them, but that the small amounts left increased plant effluent B.O.D. considerably, sometimes as much as 400 per cent. K. W. Cosens

Instrumentation In Sewage Works Practice. By H. M. Brown and A. Linford. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 104 (1950).

A rather complete discussion is given on fluid flow meters for both open channel and closed conduit conditions. The authors suggest that, for an activated sludge plant treating 2 m.g.d. or more, flow meters should be located at the following strategic points: Venturi tube on low-lift pump discharge, Venturi flumes on each grit chamber discharge, Venturi tubes on inlets to each primary settling tank, Venturi tubes on each aeration tank effluent line, Venturi tube on return activated sludge line, sludge meters on sludge lines from storm settling tanks, primary, and final settling tanks, as well as on the digested sludge line, and a gas meter on the digested gas outlet line.

It is brought out that with low hydraulic losses it is often difficult to split flow evenly to multiple tanks operated in series by means of symmetrical piping or channel inlets, and that overloading of some tanks will occur if meters are not installed on individual tanks.

Advantages and disadvantages of Venturi tubes and orifice plates are discussed. It is suggested that gas meters usually could be of the integrating type only. Caution is given regarding corrections for changes in density of the fluid being measured (for example, air and the effect on density caused by temperature changes).

In connection with open channel flow measuring devices, weirs are not discussed, because they are more commonly understood. A very thorough discussion is given of the Venturi flume. Methods of installing, shapes of flumes, hydraulics of flumes, and flow head transmission are discussed. The many advantages of the Venturi flume are emphasized.

Methods of transmission of flow measurements from primary flow devices to indicators and recorders are discussed.

An unusual regulating valve or device for preventing excess sewage flow from entering the plant, thus diverting storm flows, is presented. The regulator is available in sizes from 4 in. to 54 in. and is designed to allow flows to vary up to 3 times the dry weather flow and to bypass all excess flows. The successful operation of this regulator requires that it be installed in connection with a Venturi flume. K. W. Cosens

Unusual Features of Sewage Treatment at Nottingham. By A. R. Stone and W. E. Abbott. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 116 (1950).

From 1877 to 1936 sewage irrigation or sewage farming was practiced with limited to unsatisfactory results. In 1936, detritus tanks, screening, and primary settling, with sludge disposal in lagoons, were provided.

It was intended that sludge would be drawn into the lagoons, allowed to dry, and then be plowed into the soil.

This method was very unsatisfactory. Later a greater area of land was secured and it was possible to allow the sludge to dry 4 months, to a 3-in. thick layer, at which time it was successfully plowed into the soil with tractors. Crops follow this in season.

A considerable amount of study was reported on efforts to use the green algae as an aid in the stabilization of settled sewage. An attempt was made to correlate the plant work with a parallel laboratory investigation. It was concluded that the presence of algae prevented the escape of odors causing nuisance in the plant area.

The authors carefully brought out the fact that B.O.D. could easily be increased or decreased by the photosynthesis process. The organic material developed by photosynthesis being decomposed during the night, when oxygen production was at a standstill, might increase B.O.D., although daytime oxygen production would reduce B.O.D. K. W. Cosens

Burgh of Motherwell and Wishaw—Clyde Park Sewage Purification Works. Anon. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 126 (1950).

The plant is designed for a flow of 1.5 m.g.d., serves a population of 26,400, and consists essentially of screening, grit removal, storm-water tanks, pre-aeration tanks, primary settling, compressed air-type activated sludge, final settling, sludge digestion, and sludge drying beds.

Screens have a clear opening of 1 in. and are equipped with a raking mechanism and disintegrator pump, which returns the ground screenings to the flow. The pump is activated by a float mechanism, which operates on the head loss through the screen. A manually raked by-pass screen is provided.

Conventional detritus removal is provided, grit being loaded into narrowgage cars and thence into trucks and carted away.

Storm-water or balancing tanks capable of handling three times dry-weather flow at a 1-hr. detention period, are provided. They are constructed with longitudinal ridge-furrow bottoms having sufficient bottom slope to facilitate cleaning after use. Dry-weather flow utilizes only one channel. Flow in excess of three times dry weather is by-passed to the river.

A Venturi flume measures the plant flow after passing through the storm tanks on the way to pre-aeration tanks, which are equipped with ridge and furrow bottoms. Mechanical flocculation and grease removal are accomplished in a 45-min. detention period with diffused air.

Dry-weather detention in the primary sedimentation tanks is 6 hr. Continuous scrapers move the sludge to the inlet end hoppers, where it is removed under hydrostatic head by telescoping sludge valves. Automatic scum removal is also provided.

From the primary tanks the sewage passes to ridge and furrow aeration tanks, where it is mixed with settled activated sludge. Diffused air aeration at 6 lb. per sq. in. pressure is continued for 8 hr., with the sewage then passing through an aerated channel to the final settling tanks, where 6-hr. detention is provided.

Activated sludge is continuously drawn from the hopper bottomed final tanks through telescoping valves and is split for return to the influent of the aeration tanks and waste to the pre-aeration tank inlet. Air lift pumps are used for returning activated sludge.

Sludge collected in the storm-water and primary tanks is pumped to the primary digester, equipped with floating cover and heated by hot-water coils, which are mounted on the rotating arm of the stirring mechanism. Partially digested sludge passes from the primary digester to the secondary-stage digester, which is only half as large. Approximately 3 cu. ft. per capita sludge digestion space is provided. Supernatant is drawn off the secondary digester; gas is collected in the primary stages and used at the plant for heating and lighting.

Sludge is metered as drawn from the secondary digester to open sludge drying beds, which provide 1.5 sq. ft. per capita. The 9-in. sludge layer dries to a 3-in. cake and is disposed of as manure. Sludge bed drainage is returned to plant inflow by float-controlled pumps.

Air compressors are vertical twin-cylinder low-pressure machines driven by electric motors. Sludge pumps are of the single-plunger type. Total plant cost was approximately £140,000.

K. W. Cosens

Rapid Gravity Sand Filtration of Humus Effluent Without Coagulants. By J. J. C. Heynike and J. W. H. Westof. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 135 (1950).

Pilot plant studies made at the Ancor sewage treatment works of the town of Springs, South Africa, are reported. After trials of filtering humus tank effluent through pressure filters at 10-ft. and 6-ft. loss of head, it was decided to continue the experimental work using gravity filters and a head loss of 6.75 ft.

The filtering media consisted essentially of strata as shown in the following:

Depth and Description	Screen Size	
	Passing	Retained on
2 in. under drain coarse stone	1½ in.	¾ in.
3 in. pebbles	½ in.	¼ in.
3 in. coarse sand	¼ in.	$\frac{1}{16}$ in.
6 in. sand	No. 10[1]	No. 20[1]
2 in. stone	1½ in.	¾ in.
20 in. sand	No. 20[1]	No. 30[1]

[1] Tyler mesh.

Results of one of the experimental runs are reported as follows. After 1.5-hr. detention, humus tank effluent was applied to the sand filter at a rate of 155 gal. (Imp.) per sq. ft. per hr.; suspended matter was reduced 80 per cent (from 25 to 5 p.p.m.); 4-hr. oxygen absorbed was reduced 30 per cent (from 9.6 to 6.7 p.p.m.); and 5-day B.O.D. was reduced 49 per cent (from 19.7 to 10.1 p.p.m.).

Considerable work was done in developing backwashing methods and rates. The authors concluded that backwash rates of 22 to 25 gal. (Imp.) per sq. ft. per min., providing 40 to 50 per cent sand expansion, were satisfactory. At this rate, 50 to 60 gal. of filter effluent per square foot of filter was needed for satisfactory backwashing. Mud balls formed and could be removed only with the use of air in the backwashing process.

The effect of chlorination of humus tank effluent before filtration is suggested as a subject of further investigation.

K. W. Cosens

Effect of Urea on the Determination of Ammonia in Sewage. By S. H. Jenkins. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 149 (1950).

The following conclusions to a study of the subject are given:

1. Normal domestic sewage, freshly sampled at a sewage works, may contain an appreciable amount of urea.

2. The urea is hydrolyzed to ammonia completely by incubating the sewage for 2 hr. at 33° to 35° C. with 5 urease tablets per liter of sample.

3. The urea in normal domestic sewage is also hydrolyzed by allowing the samples to stand for 24 hr. at room temperature without added urease.

4. Secondary sedimentation tank effluent and final effluent gave the same ammoniacal nitrogen figures before and after incubation at 33° C., before and after keeping for 24 hr. and before and after incubation with urease.

5. Evidently, trade wastes (acid) inactivated the added urease, as well as the normal urease of the sewage.

6. Urea in sewage is partly recovered as ammonia by distillation.

7. Fresh samples of sewage should not be analyzed for ammoniacal nitrogen until after standing 24 hr., unless the analyst is interested in the urea content. Precautions should be taken to be sure that urea is hydrolyzed, otherwise the results of ammoniacal and organic nitrogen content are affected. Ordinarily, sewage samples contain no urea. Fresh sewage which does not show an increase in ammoniacal nitrogen content on storage or on incubation with urease and has had a presistently high organic nitrogen content may be suspected of industrial wastes inhibiting the hydrolysis of urea.

K. W. Cosens

Observations on the Determination of Nitrite in Sewage Effluents. By L. Klein. *Jour. and Proc.*, Inst. Sew. Purif., Part 2, p. 153 (1950).

Comparison of nitrite determinations in sewage effluents by the Griess-Ilosway and m-phenylene diamine methods showed there was good agreement between the two methods. Concentration of the chloride ion, affected by adding NaCl, up to 500 p.p.m. had little effect on the results; even up to 10,000 p.p.m. the effect was small. The amount of chloride ion

normally present in sewage effluents can have no significant effect on the determination of nitrite by either method.

K. W. COSENS

Annual Report—1950–51: Delaware Water Pollution Commission. 12 pp. (1951).

Following brief reviews of the field and laboratory operations and the publicity campaign being carried on by the Commission, the report outlines the major points of the Wilmington-New Castle County trunk sewer project, on which construction began in March 1951. Tentative completion date for the entire project is late in 1954.

The project consists basically of 5 large interceptors in Wilmington and 7 long trunk sewers to various developed areas in the county. Approximately 65 per cent of the population and 45 per cent of the industry of the state are located in the area, which comprises less than 10 per cent of the state's area.

The waste treatment plant, in the Wilmington area, will be of primary treatment type, designed to handle an average of 40 m.g.d. and a maximum storm flow of 160 m.g.d. Present estimated population is 150,000; ultimate population, 250,000. Average sewage flow consists of 27 per cent domestic, 40 per cent large industrial, 13 per cent large commercial, and 20 per cent infiltration. Plant processes will include settling, separate digestion, sludge elutriation, and land-fill disposal of sludge.

Total cost of the project is estimated at $14,500,000, of which about $9,000,000 is to be borne by Wilmington and the remainder by the county.

H. P. ORLAND

Design for Extensions to the Rodbourne Sewage Works, Swindon. BY H. T. COGGON. *Surveyor*, **110**, 585 (Sept. 15, 1951).

At present, the town of Swindon has two sewage works, both of which are overloaded. Their combined capacity is 2 m.g.d., whereas the dry-weather flow has been found to be 2.8 m.g.d. New sewers are planned to bring the entire flow to one of the plants, which is to be enlarged and modernized.

The improvements are to be accomplished in stages, the first of which is to bring the plant capacity to 3 m.g.d. The revised design for this stage of the work is described in detail. The main detritus tanks will treat 16 m.g.d. Storm water in excess of this flow will pass to storm-water tanks and thence to underdrained storm-water land. Beyond the detritors, flows above 9 m.g.d. will be diverted to the storm-water system. The flow for full treatment will pass through bar screens, a grit removal tank, and a measuring flume, to the sedimentation tanks. The settled sewage will then be treated on recirculating trickling filters, followed by humus tanks.

Sludge treatment will be by heated digestion followed by air drying.

The second stage of the improvements will increase the capacity for full treatment to 5 m.g.d., and permit all flow below 30 m.g.d. to pass through the grit-removal tank before diversion to the storm-water system. M. C. RAND

Summary Report on Water Pollution: Pacific Northwest Drainage Basins. U. S. Public Health Service Publ. No. 87 (1951).

Sewage and waste treatment projects estimated to cost $123,000,000 are recommended at 398 municipal and industrial locations in the Pacific Northwest Drainage Basins by the fifth in a series of summary reports on pollution control in the major drainage areas of the United States. The area covered by the report includes the states of Washington, Oregon, and Idaho, as well as parts of Nevada, Utah, Wyoming, and Montana.

A total of 434 municipalities, institutions, and sanitary districts discharge sewage and industrial wastes with a population equivalent of 3,000,000 through public sewer systems, with not all of the industrial wastes discharged to city sewers known. There are also 824 known separate industrial waste outlets in the basins, of which 211 produce organic wastes with a population equivalent of 13,000,000, organic wastes of undetermined population equivalent are produced by 432 others, and 181 produce inorganic wastes or wastes of undetermined type.

Incomplete data indicate a need for 398 sewage and wastes treatment projects, including 331 for municipalities and 67 for industries. Many of the municipal plants would also treat industrial effluents. Municipalities need 231 new treatment

plants, enlargements or additions to 43, and replacement of 57 plants. Industry needs 54 new plants, enlargements or additions to 7, and 6 replacements. No projects are needed at 128 municipal and 58 industrial locations, but project needs are undetermined for 24 municipalities and 699 industries.

Initial cost of the required municipal plant projects is estimated at $93,000,000. Another $30,000,000 is estimated as the initial cost of industrial waste treatment needs, exclusive of industrial wastes now discharged to municipal sewer systems.

H. P. ORLAND

Stream Pollution Resulting from the Use of Organic Insecticides. BY L. A. YOUNG AND H. P. NICHOLSON. *Prog. Fish-Cult.*, **13**, 4, 193 (1951).

In August, 1950, fish kills occurred in at least 15 streams in the Tennessee River Valley of Alabama. Investigations indicated that the kills resulted indirectly from the application of insecticides for the control of cotton insects. These are the first instances known to the authors in which extensive pollution of streams has resulted from the use of insecticides in agriculture.

In 1950, insecticides containing toxaphene, benzene hexachloride, DDT, and aldrin became widely used on cotton for the first time in this area. Frequent showers during the dusting season necessitated re-application of the insecticides at shorter than normal intervals, with resultant large concentrations on the soil of cotton fields. Several heavy rains in August presumably washed into the streams sufficient insecticide to eradicate fish life from some streams, and cause lighter kills in others. The kills followed shortly after heavy rains and were associated with high water levels and increased turbidities.

Stream pollution resulting from employment of organic insecticides in agriculture may become an increasingly serious problem. The insecticides now employed for the control of cotton insects, and the recommendations for their use, should be re-evaluated in an attempt to prevent further similar occurrences. Every effort should be made to prevent individual farmers from applying insecticides in excess of the quantities recommended.

H. P. ORLAND

Sewage Works Design: Some Important Considerations. BY JOHN HURLEY. *Surveyor*, **110**, 727 (Nov. 17, 1951).

Rather than comprehensively discussing all phases of sewage works design, the author attempts only to point out certain considerations which he feels often receive insufficient attention.

It should be considered prerequisite for the designer to know precisely the effluent quality required of his plant. All pertient local conditions, as well as legislation, should be considered in setting the standards and in devising means to meet them. The site, area, and levels available for the proposed plant may be decisive in selecting the type of treatment. It is necessary to locate the plant so as to avoid complaints of fly and odor nuisance.

The question of what effluent quality a plant can produce is no more important than that of how consistently it can produce that quality. Occasional discharge of a poor effluent can be most damaging to the receiving stream. Therefore, facilities should be provided for continuing adequate treatment while necessary maintenance is performed. In this connection it is also suggested that all storm flows up to three times the dry-weather flow should be treated.

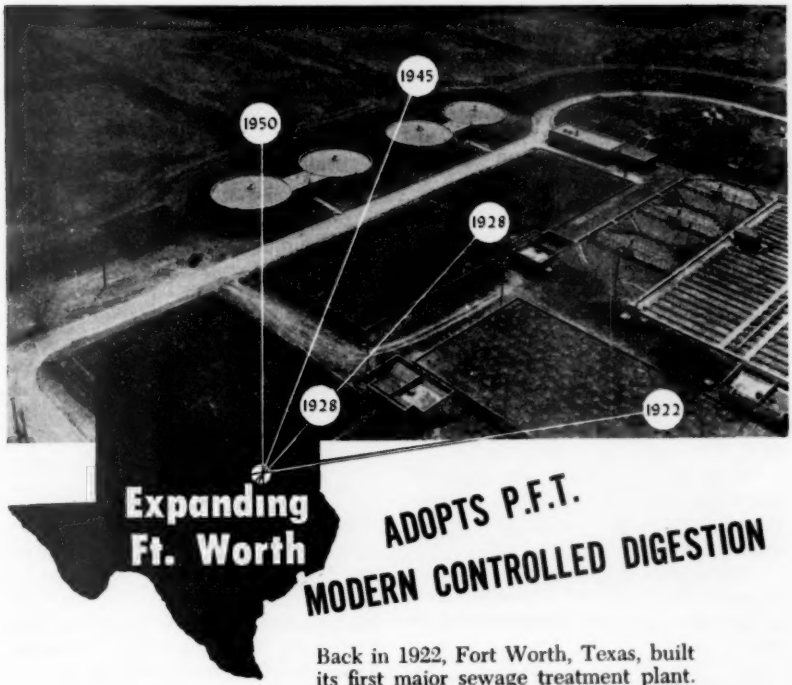
The designer should consider ease of operation, maintenance, and supervision, at night and in bad weather especially. He should consult with experienced operating personnel on these points if he is not himself experienced.

It frequently may be desirable to equip the plant for experimentation with different modes of operation.

The effect of trade wastes must be considered in the design, and it is not always sufficient to evaluate their strength by the criteria used for sewage. Pretreatment at the source of such wastes must also be taken into account.

Sludge liquor, surplus activated sludge, and such drainings are usually returned to the raw sewage. Improved results may be realized if they are pretreated before recirculation.

M. C. RAND



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Proceedings of Member Associations

MONTANA SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 8th Annual Meeting of the Montana Sewage and Industrial Wastes Association was held at the Northern Hotel, Billings, Mont., April 10, 1952. Registration totaled 86.

Papers presented were as follows:

"Critical Materials Situation," by Gerald Arnold, Director, Water Resources Division, NPA, Washington, D. C.

"Sewage Treatment for Montana Municipalities," by E. R. Dodge, Head, Dept. of Civil Engineering, Montana State College, Bozeman.

"Utilization of Industrial Wastes," a panel discussion by representatives of industry and state regulatory agencies.

The meeting concluded with an inspection tour of the waste disposal facilities of the Carter Oil Company refinery and the Billings sewage treatment plant.

Officers elected for the ensuing year were:

Chairman: Sam Ward, Billings.
Vice-Chairman: Carl H. King, Chinook.
Secretary-Treasurer: H. W. Taylor, Helena.

H. W. TAYLOR,
Secretary-Treasurer

NEW YORK SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 24th Annual Meeting of the New York Sewage and Industrial Wastes Association was held at the Statler Hotel, New York City, January 17–18, 1952. Registration of members and guests totaled 285.

At the first morning's technical session the following papers were presented:

"Efficiency of Effluent Chlorination in the Metropolitan Area," by W. W. Sanderson, Chemist, State Department of Health, Albany.

"Lethal Effects of Ultrasonic Vibrations in Sanitary Engineering Applications," by J. P. Horton, Sanitary Engineer, National Research Laboratories, Oakdale, L. I.; D. E. Phinney, Research Assistant; and M. P. Horwood, Professor of Sanitary Science, Department of Civil and Sanitary Engineering, Mass. Inst. of Tech., Cambridge, Mass.

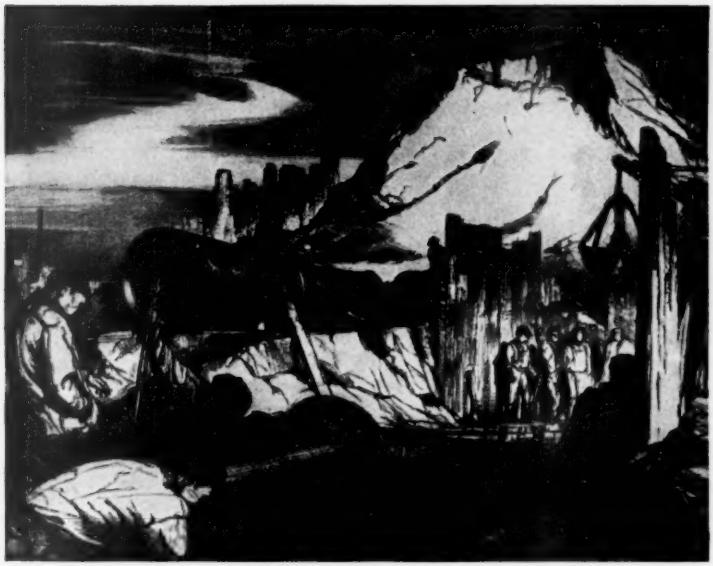
At the luncheon meeting, E. Sherman Chase, FSIWA Vice-President, was the guest speaker. The Association's Kenneth Allen Award for 1951 was presented to W. N. Torpey of New York City. The Lewis Van Carpenter Award for 1951 was presented to N. I. Kass, W. N. Torpey, A. H. Chasick, and R. C. French, all of New York City.

The afternoon technical session included a paper on "A New Approach to Sewage Works Financing," by M. M. Cohn, Editor, *Wastes Engineering*, and City Manager, Schenectady. A panel discussion titled "Industrial Waste Treatment is a Point of View," was presided over by J. M. Brown, Sanitary Engineer, Lederle Laboratory Division, American Cyanamid Company, Pearl River. Panel participants and the titles of their discussions were as follows:

"The Federal Engineer Looks at Waste Treatment," by Hayse H. Black, Chief, Industrial Waste Section, Survey and Investigation Branch, U. S. Public Health Service, Cincinnati, Ohio.

"The State Engineer Looks at Waste Treatment," by R. D. Bates, Chief,

(*Continued on page 328a*)



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"The Chemical Engineer Looks at Waste Treatment," by C. H. Nichols, Senior Chemical Engineer, Engineering Division, Merck & Co., Rahway, N. J.

"The Sanitary Engineer Looks at Waste Treatment," by Thomas J. Powers, Superintendent of Waste Disposal, Waste Disposal Division, Dow Chemical Co., Midland, Mich.

Following the close of the afternoon's technical session the Water and Sewage Works Manufacturers Association was host to a social hour.

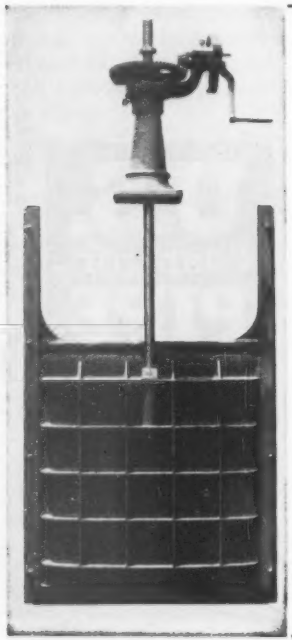
The technical session on the second morning opened with a motion picture developed by the Ace Pipe Cleaning Contractors, Inc., Kansas City, Mo., and presented by E. B. Headden of Essex Fells, N. J. The picture dealt with the 1951 "Kansas City Flood and Sewer Cleaning Experience." The session closed with an Operator's Round Table led by U. T. Mann, Superintendent of the Ley Creek plant, Syracuse. Assisting Mr. Mann were C. R. Velzy, Consulting Engineer, New York City; L. Gallagher, Worthington Pump and Machinery Corp., Harrison, N. J.; and John C. McDonald, Superintendent of Sewage Treatment, Springfield, Mass.

Officers elected to serve during 1952 were as follows:

President: W. R. Schreiner, Glens Falls.
Vice-President: H. A. Faber, New York City.
Treasurer: J. C. Brigham, Albany.
Secretary: R. C. Sweeney, White Plains.

The meeting closed with an inspection trip to New York City's new Hunts Point sewage treatment plant.

R. C. SWEENEY,
Secretary



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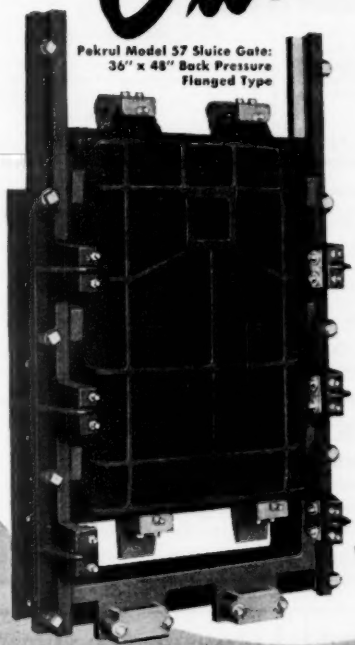

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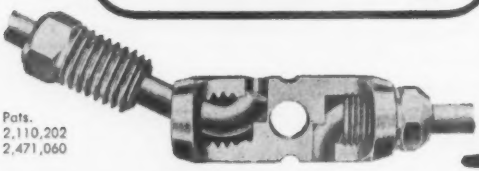

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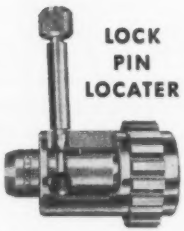

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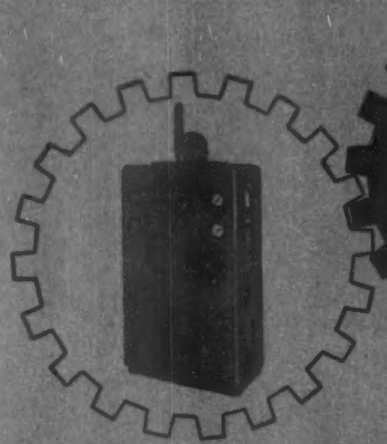

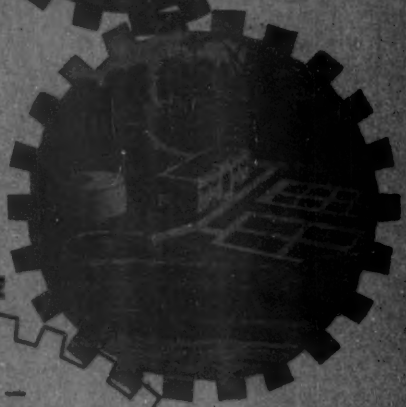

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